(4)

CR 89.011

July 1989

NCEL Contract Report

An Investigation Conducted by
N. Sathialingam and B. L. Kutter

Sponsored by Office of Naval Research
Arlington, VA 22217-5000

The Effects of High Strain Rate and High Frequency Loading on Soil Behavior in Centrifuge Model Tests

ABSTRACT A review of available data regarding the influence of time effects on the stress-strain behavior of soils is presented. During simulated earthquake loading, centrifuge models are expected to deform at strain rates on the order of 100 to 1000% per second, and at frequencies between 20 Hz and 1000 Hz. These strain rates and frequencies are outside the realm of conventional laboratory tests, so it is difficult to accurately determine the strain rate effects during dynamic centrifuge tests. To supplement ~~the data from~~ laboratory tests, data from centrifuge tests employing modeling of models are also cited. It is found that strain rate and high frequency effects are small for sands. For clays there is a significant trend of increasing strength and stiffness as the rate of strain increases. Sample nonuniformity is a serious problem for tests conduted at very high frequencies and high strain rates. Some data show that the strain rate effect becomes more important at about 100% per second. This increase of rate effects may be related to errors associated with sample uniformity. The review of centrifuge tests results did not indicate that strain rate and high frequency effects was a serious problem.

NAVAL CIVIL ENGINEERING LABORATORY PORT HUENEME CALIFORNIA 93043

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER CR 89.011	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) The Effects of High Strain Rate and High Frequency Loading on Soil Behavior in Centrifuge Model Tests		5. TYPE OF REPORT & PERIOD COVERED Final Oct 1987 - Sep 1988
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) N. Sathialingam B.L. Kutter		8. CONTRACT OR GRANT NUMBER(s) N62583-88-P-1412
9. PERFORMING ORGANIZATION NAME AND ADDRESS University of California, Davis Davis, CA 95616		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS PE-N62583 WU-YR02303A
11. CONTROLLING OFFICE NAME AND ADDRESS Naval Civil Engineering Laboratory Port Hueneme, CA 93043-5003		12. REPORT DATE July 1989
		13. NUMBER OF PAGES 61
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office) Office of Naval Research Arlington, VA 22217-5000		15. SECURITY CLASS. (of this report) Unclassified
		15a. DECLASSIFICATION DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution is unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Geotechnical centrifuge, strain rate, soil dynamics, soil constitutive models

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A review of available data regarding the influence of time effects on the stress-strain behavior of soils is presented. During simulated earthquake loading, centrifuge models are expected to deform at strain rates on the order of 100 to 1000% per second, and at frequencies between 20 Hz and 1000 Hz. These strain rates and frequencies are outside the realm of conven-

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered) tional laboratory tests, so it is difficult to accurately determine the strain rate effects during dynamic centrifuge tests. To supplement the data from laboratory tests, data from centrifuge tests employing modeling of models are also cited. It is found that strain rate and high frequency effects are small for sands. For clays there is a significant trend of increasing strength and stiffness as the rate of strain increases. Sample nonuniformity is a serious problem for tests conducted at very high frequencies and high strain rates. Some data show that the strain rate effect becomes more important at about 100% per second. This increase of rate effects may be related to errors associated with sample uniformity. The review of centrifuge test results did not indicate that strain rate and high frequency effects was a serious problem.

TABLE OF CONTENTS

1.0 INTRODUCTION

This report is written in two parts. The first part deals with monotonic strain-rate effects. The second part deals with the effect of high frequency loading on soil behavior. High frequency loading at a high strain amplitude is a special case of high strain rate loading in which the direction of loading is reversed and repeated. The main goal of this report is to assess the effect of high frequency loading on the mechanical properties of soils. Due to the sparsity of cyclic loading data available at the frequencies of interest, however, the effects of loading rate in monotonic shear tests is discussed in some detail.

In centrifuge modeling, a small scale model is subjected to increased acceleration field in order to obtain the same stress levels in model and prototype. Since the models under consideration are smaller than the prototype by an arbitrary scale factor, n, the events will occur more rapidly in the model than in the prototype.

For consolidation, a diffusion process, the length of a drainage path is reduced by the factor n in the model. Therefore, events occur n^2 times faster in the model than in the prototype. In the case of dynamic model testing such as the simulation of earthquake loading, timing of events is related by $t_p = nt_m$, where the subscripts p and m denote prototype and model. The rate of change of stress and strain in the models are increased in order to satisfy stress and strain similarly.

The predominant range of frequencies observed in real earthquakes is between about 0.2 and 10 Hz, with most of the energy (the peak in the velocity spectrum) occurring at about 1 to 2 Hz for soil sites. A 1 Hz prototype shaking frequency f_p is modelled in an n g centrifuge test by a $f_m = nf_p$ model frequency. The frequencies of interest for modeling earthquake effects in the centrifuge are in the range of 0.2 n to 10 n, where n typically varies between 20 and 100. Thus it is conceivable that loading frequencies up to 1000 Hz are important, though most of the energy would be below about 200 Hz (for a test at n = 100).

Since length and displacements scale according to $\ell_m = \ell_p/n$, and accelerations scale according to $a_m = \ell_m/t_m^2 = n(\ell_p/t_p) = na_p$ (in dynamic tests) it is apparent that $t_m = t_p/n$. From dimensional analysis it is also apparent that $v_m = \ell_m/t_m = v_p = \ell_p/t_p$, that is, velocities are the same in model and prototype.

To simulate an 0.25 g 1 Hz prototype earthquake pulse at n = 100 would require a 25 g, 100 Hz pulse in the model. An 0.25 g, 1 Hz pulse would involve a peak velocity of

$$v_p = \frac{a_p}{2\pi f_p} = \frac{0.25(9.81\ m/s^2)}{2\pi(1\ Hz)} = 0.39\ m/s$$

likewise in the model

$$v_m = \frac{a_m}{2\pi f_m} = \frac{25(9.81\ m/s^2)}{2\pi(100\ Hz)} = 0.39\ m/s$$

The peak displacement of a sinusoidal pulse would be

$$d_p = \frac{a_p}{(2\pi f_p)^2} = 0.062\ m$$

$$d_m = \frac{a_m}{(2\pi f_m)^2} = 0.00062\ m = 0.62\ mm$$

The order of magnitude of the strains that would be reached in the model and prototype can be estimated by $\epsilon = d/h$, where h is a characteristic dimension of the soil layer undergoing a relative displacement, d.

For a prototype layer with $h_p = 10$ m a representative strain that would result from a 0.25 g, 1 Hz pulse would be

$$\epsilon_p = \frac{d_p}{h_p} = \frac{0.062 \text{ m}}{10 \text{ m}} = 0.62\%$$

in a model with $n = 100$, the thickness h_m would be $h_p/100 = 0.1$ m and the corresponding strain would be

$$\epsilon_m = \frac{d_m}{h_m} = \frac{0.62 \text{ mm}}{0.1 \text{ m}} = 0.62\%$$

So, as expected, strains are the same in model and prototype.

The strain rates, however, would be

$$\dot{\epsilon}_p = \frac{v_p}{h_p} = \frac{0.39 \text{ m/s}}{10 \text{ m}} = 0.039/\text{s} = 3.9\%/\text{s}$$

$$\epsilon_m = \frac{v_m}{h_m} = \frac{0.39 \text{ m/s}}{0.1 \text{ m}} = 3.9/\text{s} = 390\%/\text{s}$$

Thus expected strain rates in the dynamic centrifuge model tests are in the range of 100 to 1000%/second.

Cyclic triaxial and simple shear tests have been reported in the literature at these strain levels, but not at typical model frequencies. Resonant column tests can be conducted at these frequencies, but not to the above strain levels. Neither of these types of tests has typically been conducted at the appropriate strain rates. Some monotonic tests have been conducted at strain rates appropriate to dynamic centrifuge tests, but even in monotonic tests difficulties arise due to non-uniform deformation caused by the limited rate at which stress waves can travel through a specimen.

It has been recognized for a long time that the soil properties such as moduli and shear strength are affected by the rate of loading. For the most part, the existing data in the literature is predominantly obtained for rates of loading that are smaller than those which may occur in dynamic centrifuge model tests. An attempt is made in this report to extrapolate the data from the literature to rates of interest.

2.0 THE EFFECTS OF HIGH STRAIN-RATE LOADING ON SOIL BEHAVIOR

2.1 Rate Process Theory

Deformation and shear failure of soils involves time-dependent rearragnement of matter. As such these phenomena are amenable for study as "rate processes" through application of the theory of absolute reaction rates (Glasstone, Laidler and Eyring, 1941).

The basis of rate process theory is that atoms, molecules and/or particles participating in a time dependent flow or deformation process, termed "flow units" are constrained from movement relative to each other by virtue of energy barriers separating adjacent equilibrium positions. Based on these principles, Mitchell (1976) derived a relationship for strain rate as shown below.

$$\dot{\epsilon} = x \frac{kT}{h} \exp(- \frac{\Delta F}{RT}) \exp (\frac{D\lambda}{4SkT})$$

where x = proportion of successful barrier crossing and the corresponding displacement

k = Boltzmann's constant

T = absolute temperature

h = Planck's constant

ΔF = activation energy

R = universal gas constant

D = deviator stress under triaxial condition

λ = distance between successive equilibrium positions

and S = "flow units" per unit area

If the maximum shear stress τ is substituted for the deviator stress D in the above equation, taking logarithms would give

$$\ell n\ \dot{\epsilon} = \ln\left(x\,\frac{kT}{h}\right) - \frac{\Delta F}{RT} + \frac{\tau\lambda}{2SkT}$$

Assuming $(x\ kT/h) = B$ is constant (Mitchell, 1964).

$$\tau = \frac{2S}{\lambda N}\Delta F + \frac{2SkT}{\lambda}\ \ell n\left(\frac{\dot{\epsilon}}{B}\right)$$

Since there exists a unique relationship between bonds per unit area and effective stress for all soils, a relationship for "S" could be written as follows

$$S = a + b\sigma_f'$$

where a and b are constants and σ_f' is the effective stress on the shear plane. Thus the equation for "τ" becomes

$$\tau = \left(\frac{2a\Delta F}{\lambda N} + \frac{2akT}{\lambda}\ \ell n\ \frac{\dot{\epsilon}}{B}\right) + \left(\frac{2b}{\lambda N}\Delta F + \frac{2bkT}{\lambda}\ \ell n\left(\frac{\dot{\epsilon}}{B}\right)\right)\sigma_f'$$

This is of the same form as the Coulomb equation for strength.

$$\tau = c + \sigma_f' \tan\phi.$$

By analogy (Mitchell, 1976)

$$c = \frac{2a\Delta F}{\lambda N} + \frac{2bkT}{\lambda} \ln(\frac{\dot{\varepsilon}}{B})$$

and $$\tan\phi = \frac{2b}{\lambda N}\Delta F + \frac{2bkT}{\lambda}\ln(\frac{\dot{\varepsilon}}{B})$$

This shows that, all other factors being equal, the shearing resistance should increase linearly with the logarithm of the rate of strain.

2.2 Literature Review

2.2.1 Introduction

As early as the nineteenth century, Vicat (1833) and Collin (1846) recognized that strain rate had an effect on the undrained shear strength of cohesive soils. In addition studies of strain-rate effects on the compressive strength of clays were performed at Harvard University in the late forties. They investigated the undrained stress-deformation and strength characteristics of soils under very rapid loading and unloading (Casagrande and Shannon, 1948a, 1948b, 1949).

The findings of these early studies created a significant interest in the effect that rate of loading has on the shear strength of soils. As a result, this problem has been investigated by many authors in the following years in relation to undistrubed, remolded and compacted clays. (See, for example: Whitman (1957); Bjerrum, et al. (1958); Crawford (1959); Richardson and Whitman (1963); Perloff and Osterberg (1963); Jarrett (1967); Graham et al. (1983)).

2.2.2 Behavior of Cohesive Soils

From earlier studies (carried out at Massachusetts Institute of Technology, 1954) one pattern of behavior has emerged, as shown in fig. 1 (Whitman, 1960). It seems possible to distinguish between the behavior of the samples which fail in a brittle fashion by cracking or splitting and those samples which deform in a plastic fashion.

When a sample fails in a brittle way, the inference may be drawn that the bonds within the sample have been overcome, and once overcome have been destroyed rather completely. On the other hand, in a plastic type of failure the material remains intact.

For soils which deformed in a plastic fashion, the pattern of strain rate effect suggested some sort of viscous action. The stress-strain curves for slow and fast tests showed the same general shape, and the stress at 1% strain was increased by the same percentage by increasing the strain rate as was the stress at 10% strain. The soils which deformed in a plastic way generally showed moderate strain-rate effect, i.e.; the strength increased on the order of 50% as the time to failure was decreased from several minutes down to several milliseconds (Whitman, 1960).

The soils which broke in a brittle way generally showed larger strain rate effects - perhaps 100% or more strength increase for the same range of time-to-failure. The stress-strain curves from fast tests appeared quite different than those from slow tests, having much broader peaks. As the strain rate increased, so too did the strain at which the maximum deviator stress was reached. This pattern suggests that the bonds which hold the brittle samples together have time-dependent characteristics. Thus the amount of bonding which acts at a given strain increases as the strain rate increases and hence the higher strength.

Whitman (1960, 1970) suggests that it is primarily the level of pore pressures within a sample which determines the category into which the sample falls. When pore pressures are positive or slightly negative at failure (for tests which are conducted with confining stresses) the plastic type of behavior generally was encountered. On the other hand, where large negative values of pore pressure existed at failure, (as in the unconfined compression tests) the brittle type of failure generally occurred.

Whenever a specimen of soil is tested under small or zero confining stress, as in unconfined compression tests, the soil has strength largely because there are capillary tensions, which in turn cause increased effective stresses. In general, these capillary tensions become increasingly important as the water content decreases, although in very dry soils, they cease to be important with regard to strength.

In conclusion, Whitman (1960) has hypothesized (fig. 1) that the strain-rate effect upon strength will be greatest when negative pore pressures contribute most importantly to strength. This hypothesis is supported by various test results reported in Table 1 (Whitman, 1970). All of these results are from either unconfined compression or consolidated undrained test on undisturbed specimens of clay. For unconfined compression tests, the strain rate effect was less at low strains, such as 1/2-4%, than at failure. For consolidated undrained tests strain rate effect was the same at low strains and at failure. In addition he believes that the structural viscosity is of less importance for rate effects than the changes in the excess pore pressure.

Richardson, et al. (1963) carried out tests using strain rates 1%/min and 0.002%/min and concluded that the mechanism for the effect of strain rate is different for small strains and large strains (Table 2).

At large strains, the relationship between void ratio and effective stress apparently is strain rate dependent; that is, as the strain rate is increased, a larger effective stress is required to hold the soil at any given void ratio. At the faster strain rates, adjacent soil particles find it more difficult to move relatively, and, unless restrained by increased effective stress, will tend to ride up over one another. The effect can be thought of either as an increased resistance to compression or as an increased tendency to dilate. In either case the result is the same: increasing the strain rate applied to a saturated soil means larger effective stresses and consequently greater shear resistance.

At small strains, increasing the strain rate caused an increase in (σ_1'/σ_3') where σ_1' and σ_3' are the major and minor principal effective stresses, respectively. This result is interpreted to imply a strain-rate effect upon resistance to distortion, whereas the strain rate effect at large strains is more related to volume change phenomena.

Figure 2 shows the deviator stresses which developed in four different tests following a sudden increase in strain rate from 0.002%/min to 1%/min (Richardson, et al., 1963). Average curves from the tests with a steady-strain rate have been superimposed. It can be observed that,

a) The deviator stress increases sharply to a level greater than that which was achieved at the same strain in fast tests with a steady strain rate.

b) Following the peak, the deviator stress decreased to the level achieved during steady, rapid straining.

Figure 3 shows the relationship between obliquity ratio (σ_1'/σ_3') vs. strain computed from the test results shown in fig. 2. It appears from the results that the strain rate effect upon peak deviator stress will be greater for anisotropically consolidated than for specimens consolidated under $\sigma_1' = \sigma_3'$.

Drescher (1966) has also observed the same phenomena for the effect of abrupt variation of the strain rate upon the strength characteristic of a cohesive soil, for strain rates of 2.1%/sec - 8.3×10^{-4}%/sec. It is interesting to note that Dreshcher's (1966) experimental results have no distinct failure strength even after 15% strain as shown in fig. 4.

Perloff, et al. (1963) have indicated that the undrained strength of a soil can be expressed as a function of the overconsolidation ratio and the relative strain rate. The "Grundite" clay which has the clay mineral illite as its chief constituent was used in this study. The classification properties are shown in Table 3. For the soil tested the explicitly empirical relationship could be written as follows:

$$\left(\frac{\sigma_1 - \sigma_3}{\sigma_c}\right) = a \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_o}\right)^b \left(\frac{\sigma_p}{\sigma_c}\right)^s + d \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_o}\right)^f$$

where ε_o is the base value of strain rate equal to one percent per hour, σ_c is the consolidation pressure immediately prior to testing, σ_p is the maximum preconsolidation pressure and a, b, s, d, and f are dimensionless constants of the equation. These constants could be obtained from the laboratory calibration process. Table 4 gives the values of these constants for the clay studied.

It is implied by the above equation that the effects of stress history and strain rate on shear strength of soil are interdependent. In conclusion, Perloff, et al. (1963) pointed out that in order to consider one of the factors separately, it is necessary to hold the other constant. If this is not done, it is obvious that any results obtained would be liable to misinterpretation.

On the other hand, Schmid, et al. (1965) suggested that the shear strength of a clay soil could be expressed as

$$\frac{(\sigma_1-\sigma_3)}{2} = A\ e^{-D(w-w_0)}$$

where w_0 - reference water content

A - reference shear strength depending on w_0

and D - the slope of the line showing the relationship between water content and the logarithm of the shear strength.

They suggested to choose the water content at the plastic limit for w_0. Since the remoulded shear strength at the plastic limit of a soil is constant, the resulting value of A would be simultaneously a measure of the sensitivity to remoulding.

For the shear strength of a clay soil to be time-dependent, the parameters A and D should be functions of the load duration or the strain rate. To verify this intuition, Schmid, et al. (1965) carried out load controlled triaxial tests in two clays, one predominantly kaolinite (Grantham) and one illite (Grundite). The test specimens were prepared from a common batch of saturated clay and were consolidated in the triaxial cells under different confining pressures. The specimens were then subjected to shear stress while drainage was prevented. The stress was applied in uniform load increments that were held constant throughout the load interval. Load intervals used in the experiment are 10 seconds, 1 minute, 10 minutes and 90 minutes. The results are as shown in figs. 5 and 6.

Even though there is some scatter, the data clearly show that parameters A and D are functions of the load interval or of the total time to failure (Schmid, et al., 1965). The scatter is more pronounced in Grantham clay than in the Grundite. This was attributed to the less skill and experience Schmid, et al. had in choosing the proper stress increments for Grantham clay, which was tested first.

Tests on Grundite clay at various constant strain rates by Perloff and Osterberg (1963) were evaluated by Schmid, et al. (1965) on the same basis and also show a clear dependence of A on the strain rate (fig. 7). However, the data (Perloff, et al., 1963) seem to indicate that the value of D changes very little, if at all, and that perhaps the only time dependent parameter is the value A.

Lefebvre, et al. (1987) pointed out that for overconsolidated clays, pore pressures generated at a given deviator stress are essentially independent of the strain rate, while the peak strength envelope is lowered as the strain rate is decreased. For normally consolidated clays, a lower strain rate results in an increase in pore pressure generation during shearing, while the peak strength envelope remains the same. However, they concluded that, from a quantitative standpoint, the increase in shear strength ratio (i.e. $c_u/(c_u$ at 1% per hr)) caused by an increase in strain rate is similar for normally and overconsolidated clays, and it is linear for at least five log cycles of strain rate (fig. 8).

Cheng (1980) has indicated from his unconfined torsional tests on clay having Liquid Limit (LL) = 61 and Plastic Limit (PL) = 28, that the undrained strength is bounded by an upper limit called the ultimate dynamic strength (S_m), which can be written as

$$S_m = S_o + \alpha$$

where S_o = static undrained strength

and α = dynamic component of the ultimate dynamic strength.

For strain rates less than 1760 %/sec, the relation between undrained strength S, and strain rate $\dot{\gamma}$, with no confining pressure could be written as

$$S = S_0 + \alpha\,(1-e^{-\beta\dot{\gamma}})$$

where β is the coefficient that gives the rate at which the strength approaches the ultimate dynamic strength. The parameter "α" seems to depend on the water content of the clay.

Some investigations of the effect of strain rate on monotonic soil properties, described above are summarized in fig. 9. Originally some of the results were expressed in terms of strength versus time to failure. By making the assumption that failure occurred at an axial strain of 10%, the results have been plotted as the normalized strength versus the approximate axial strain rate. It was then possible to add data from Yong and Japp (1969), Drehscher (1966) and Cheng (1980). This result (fig. 9) is in contrast to the conclusion obtained by Mitchell (1976) (Section 2.1) that the relationship between strength and logarithm of strain rate is linear.

Also shown in fig. 9 is a line that corresponds to the variation in strength with strain rate of 10% per log cycle of strain rate. For most of the soils it appears that below a strain rate of about 0.1% per second, the strain rate effect is less than 10% per log cycle. Above about 0.1% per second, the strain rate effect increases above 10% per log cycle. Above about 100% per second some of the results (Cheng, 1980 and Yang and Japp, 1967) suggest that the strain rate effect increases dramatically.

2.2.3 Behavior of Cohesionless Soils

Numerous rapid loading tests have been made on dry sands (Casagrande and Shannon, 1948; Schimming, et al., 1966). All of these tests have indicated that the strain rate effect in dry sands is small or negligible, that is, that there is less than 10-15% increase in friction angle between times to failure of about 5 min and 5 m sec.

Figure 10 shows the composite average result of testing three sands (Whitman, 1970) during the earlier MIT tests. These sands were standard Ottawa sand (a uniform sand of medium grain size), a fine well graded river sand, and desert alluvium from Nevada. The results show that the friction angle first decreases as the strain rate increases beyond that required for a time to failure of 5 min. Eventually the trend reverses, and there is a slight increase in strength (and hence friction angle) with increasing strain rate.

Whitman (1970) points out that there are some reasons (Whitman and Healy, 1963) for believing that these trends are simply the result of systematic errors, and hence should be ignored. On the other hand, there is also some additional evidence to suggest that the trends may be indeed correct.

The bearing capacity tests by Vesic, et al. (1965) shown in fig. 11, show first a decrease in bearing capacity (about 30% lower than the static bearing capacity) as the loading velocity increases to about 0.002 in/sec. Further increase in loading rate to 10 in/sec, resulted in a gradual increase in bearing capacity. In tests with dry sand this increase is not very pronounced, so that the bearing capacities in the fastest tests barely exceed the static bearing capacities. In tests with saturated sand, however, the increase is very pronounced, and the final bearing capacities are several times higher than the static bearing capacities. Similar results are also reported by Colp (1965) and Whitman and Luscher (1965).

Whitman (1970) has suggested a hypotheses to explain the trend shown in fig. 10. The decrease of friction angle with increasing strain rate could be attributed to the increased tendency for kinetic friction rather than static friction to govern the behavior, as the coefficient of kinetic friction is generally less than the coefficient of static friction. The increase of friction angle at very rapid strain rates can be explained on the basis that interlocking between particles becomes more effective when the particles are not given sufficient time to find the easiest path past one another. Tests by Healy (1963) have shown that sands expand more during rapid shear than during slow shear, thus confirming the increased importance of interlocking. However, he points out these hypotheses are still only speculative and that very careful work would be required to learn just how much and just why the friction angle of dry sands is affected by strain rate.

In saturated sands there can be a strain rate effect (Whitman, 1970) as much as a factor of 2 or 3 because of differences in the excess pore pressures generated at different strain rates as shown in figs. 12 and 13. This type of effect develops when a saturated sand is straining at more or less the critical void ratio for the particular level of effective stress; i.e., at large strains with loose specimens under low to moderate effective stresses (Whitman, et al., 1962; and Healy, 1962).

This strain-rate effect in saturated sands is caused by the following phenomenon. With increasing strain rate, the sand has a greater tendency to increase in volume (Healy, 1963). In order to maintain the constant volume condition involved in undrained shear of a saturated soil, this tendency must be counteracted by an increased effective stress; i.e., by a decreased pore pressure. Any possible change in the friction angle with strain rate are of small consequence (Whitman, 1970).

Dense specimens under low effective stress will simply cavitate, and so will cease to be sheared at constant volume (fig. 14). The effect of strain rate on peak strength is then small, just as in the case of dry sands.

These results have played an important role in the understanding of the effects of strain rate on strength in the case of saturated soils. However, these results are themselves not very important in practical problems because of uncertainty as to the degree of consolidation of sands during rapid loadings will mask the strain rate effect discussed here (Whitman, 1970).

2.3 Interpretation and Extrapolation of the Results Reported in the Literature

All of the results described above were obtained using triaxial specimens except those of Cheng (1980) which were obtained using a torsional apparatus. Conducting very rapid shear tests is complicated by the inertial forces that can develop during rapid deformation. In addition rapid tests do not allow time for equalization of pore water pressures. Pore pressures measured at the sample boundaries may be different from those in the center.

First, consider the effects of inertia forces. Assuming that failure occurs at 10% strain, the average stiffness of a triaxial specimen in compression to failure is $q_f/0.1$ (refer to fig. 15). In terms of the undrained shear strength, $c_u = q_f/2$, the average modulus would be a $2\ c_u/0.1$. For a soil with a density of 2000 kg/m^3 and a shear strength $c_u = 40$ kN/m^2, the velocity of a compression wave in the sample will be

$$v_c = \sqrt{E/\rho} = \sqrt{\frac{2\ c_u/0.1}{2000}} = 20 \text{ m/sec}$$

In a triaxial specimen of typically 0.1 m height, the compression wave corresponding to the average modulus would then take 0.005 s to travel the length of the specimen. At a strain rate 100%/sec, this time lag represents an axial strain of 0.5%. A strain difference of 0.5% is itself significant, but increments of stress superimposed on a high stress will send stress waves traveling through the specimen at a velocity corresponding to the tangent modulus such as the one labelled E_{tang} in fig. 15. This modulus could be an order of magnitude lower than the average modulus, E_{avg}, and could reduce the velocity of propagation of the increment of stress accordingly. Time lags between the base and top of the specimen could be expected to be several times larger than the value calculated for the average modulus. Near failure, time lags could be expected to be on the order of 0.05 sec which at a strain rate of 100%/sec corresponds to strain lag of 5%! The torsional shear tests of Cheng (1980) would also be subject to nonuniform strain since these specimens were also relatively long (7.5 cm).

Carroll (1988) performed dynamic tests on a 1.9 cm (0.75 in) diameter by 3.8 cm (1.5 in) long soil specimen in the fast triaxial shear device (FTRXD), to study the effects of inertia in the test apparatus and wave effects in the specimen. Load and displacement at the top of the specimen and load at the bottom are measured during the experiments. The test specimens were prepared with soil taken from the CARES-Dry test site located at Luke Bombing and Gunnery Range in Arizona. It is classified as SC (clayey sand) in the Unified Soil

Classification System with a liquid limit of 36%, plasticity index of 19% and 33% fines.

Figure 16 shows the measured load versus time for the three tests (with different continuing pressures) carried out at 536%/sec strain rate. The load recorded by the upper load cell in these tests was 12-15% higher than that for the lower load cell. For an increased strain rate of 7500%/sec and a confining pressure of 200 psi, the load-time relationship is as shown in fig. 17 where the difference between upper and lower load cells is around 25%. The observed oscillation of the upper load cell was attributed to the coincidence of natural period of the loading cell with the loading rate. When this was repeated with another loading cell of lower natural period similar effects were not observed (Carroll, 1988).

It should be noted again that the above differences in loads were observed in a 3.8 cm long soil specimen. Therefore one could envisage the amount of non-uniformity that can exist in more typical 7.5 cm and 10 cm long samples in high strain rate loading. Consequently, it is difficult to determine the actual magnitude of the increase in rate effects at high strain rates (above about 100%/sec), from the results described in section 2.2.

The results for typical strain rates encountered in centrifuge tests indicate that the shear strength of soil increase at a rate of somewhat more than 10% per log cycle of strain rate (fig. 9). The apparent increase in strain rate effects at high rates of strain might be attributed to:

1) Sample non-uniformities that develop due to the limited velocity of a stress wave in the samples, or

2) Problems with the stiffness of the test apparatus like those demonstrated by Carrol (1988).

3.0 THE EFFECTS OF HIGH FREQUENCY LOADING ON SOIL BEHAVIOR

3.1 Introduction

It has long been recognized that when a soil is subjected to repeated strain cycles it undergoes irreversible structural changes and a degradation in elastic and plastic properties (i.e.: stiffness and damping) and strength. The variation of these properties during cyclic loading (or earthquake pulses) depends on loading frequency, cyclic strain amplitude, and other factors.

A representative loading frequency and earthquake acceleration associated with the response of soil deposits during earthquakes would be 1.5 Hz and 0.25 g. To model a 1.5 Hz loading frequency at model scale of 80, the required loading frequency for the centrifuge tests would be 120 Hz. In addition an 0.25 g prototype earthquake at 80 g requires a 20 g model "earthquake" acceleration.

The data that exist in the literature describing the effect of loading frequency and cyclic strain amplitudes on the behavior of soil are summarized in this report. For the most part, however, this data addresses either

1) frequencies which are significantly smaller than the 50-1000 Hz range which is involved in dynamic centrifuge tests, or
2) cyclic strain amplitudes which are much smaller than those that occur during earthquake loading which causes significant damage.

Besides summarizing existing data, this report will attempt to extrapolate the data from the literature to high loading frequencies of interest in dynamic centrifuge testing.

3.2 Literature Review

3.2.1 Introduction

Up until now, very little had been reported in the literature regarding the variations in the strength and moduli of specific soils due to different loading frequency. A few of the papers that address these issues are Taylor and Hughes (1965), Seed and Chan (1966), Theirs and Seed (1968, 1969), Brown, et al. (1975), Khaffaf (1978), and Lefebvre, et al. (1987).

3.2.2 Behavior of Cohesive Soils

Theirs and Seed (1969) performed tests on undisturbed samples of San Francisco Bay mud (plasticity index = 45). They pointed out that the two important parameters affecting the results of any seismic loading test series are the loading frequency and the trace shape. (Stress trace shape in the case of stress controlled tests and strain trace shape in the case of strain controlled tests.) They compared the results at 1 Hz with similar tests at 2 Hz conducted by Seed and Chan (1966) as shown in fig. 18. They found that increasing the loading frequency from 1 Hz to 2 Hz causes 35 to 40 percent increase in the pulsating stress required to cause failure, in a given number of cycles. (Pulsating stresses are the additional stresses induced by the earthquake.)

On the other hand, Sherif and Wu (1971) found that, in general, increasing the frequency from 1 Hz to 2 Hz has little effect on the permanent and cyclic strains and pore pressures for a given cyclic stress level, for cyclic triaxial tests on Seattle clay (Plasticity Index = 26).

From strain controlled tests on a silty clay, Taylor and Hughes (1965) stated that the increasing frequency of testing from 0.08 to 10 Hz was found to decrease the modulus by 17% (modulus measured after 1000 cycles of strain). This is less surprising, if it is considered that the samples were cycled 1000 times before their modulus was measured. Strain controlled tests at high frequency would be expected to mobilize larger stresses in the initial cycles than at low frequency. More work would thus be absorbed by the specimens loaded at high frequency and they would be expected to degrade more quickly. It is possible that the moduli during the first cycle would be greater at higher frequencies, and that degradation is faster at higher frequencies.

Brown, et al. (1975), considering the results from undrained stress controlled tests on Keuper Marl concluded that there is no significant frequency effect over the range of 0.01 to 10 Hz. But it should be noted that though the Keuper Marl was tested under relatively large one way stress cycles (up to 95% of the static strength), the strains in their tests were typically very small (on the order of 2% strain). Because their strain cycles were of low amplitude, the strain rates imposed on the specimens were relatively small. Hence their conclusion that frequency has no significant effect may not be true for all soils and all strain amplitudes.

From load controlled tests on kaolin (Plasticity Index = 35). Brewer (1972) points out in general that the pore pressure and cyclic strains at failure decrease as frequency increases between 0.1 and 10 Hz.

Khaffaf (1978) found that the higher the frequency, the lower the number of strain cycles required to induce a prescribed cyclic stress level (fig. 19). He also stated that the higher the frequency, the greater the number of stress cycles required to reach a prescribed strain amplitude (fig. 20). In addition, increasing the frequency of strain controlled tests appeared to increase the degradation of strength after cyclic loading as shown in fig. 21. From this figure it is apparent that for a 5% double amplitude strain, increasing the frequency by a factor of 100 between 0.1 and 10 Hz will cause the ratio of s_o/c_u at large number of cycles to decrease from about 0.9 to about 0.83, where s_o is the undrained shear strength after cyclic loading.

Lefebvre, et al. (1987) carried out a series of cyclic triaxial tests to study the influence of the frequency on the undrained shear strength of sensitive clays from eastern Canada. They pointed out that when comparing monotonic and cyclic undrained shear strengths, it is important to consider the difference in strain rate in the tests as well as the degradation of undrained shear strength due to cyclic loading. Significant strain rate effects in saturated clays cause the cyclic strength mobilized at high frequencies to be higher than the monotonic strength measured at standard strain rates. As a first approximation, they suggested that the cyclic strength of a clay at a given frequency could be evaluated from a monotonic test by first applying a correction for the strain rate effect to obtain the monotonic strength at the given frequency and then applying a degradation function to obtain the cyclic strength that can be mobilized for a given number of cycles.

3.2.3 Behavior of Cohesionless Soils

Peacock and Seed (1968) carried out some simple shear tests at frequencies 1/6, 1, 2, and 4 Hz on Monterey sand at a relative density of 50% and tested under a confining pressure of 5 kg/cm^2. From the results shown in fig. 22 it is apparent that for frequencies of 1 and 2 Hz, the form of the relationship between the peak pulsating shear stress and the number of stress cycles required to cause failure is essentially the same. Although data are limited, the form of this relationship is apparently the same also for the other two frequencies of 1/6 and 4 Hz. Based on the test data shown in fig. 22 Peacock and Seed (1968) concluded that Monterey sand showed only small variations due to frequency effects.

3.3 Interpretation and Extrapolation of the Results Reported in the Literature

The changes in strength and stress-strain characteristics of soil during cyclic loading is dependent on

(a) cyclic stress or strain amplitudes

(b) number of cycles

(c) loading frequency

(d) type of test (stress controlled or strain controlled)

While variations in the strength and moduli of specific soils due to variations in cyclic stress level, strain amplitude, and the number of cycles have been reported (strength by Murayama and Hata (1957); Seed (1960, 1967); Seed and Chan (1964, 1966); Theirs (1965); and Theirs and Seed (1968), moduli by Converse (1961); Parmelee, et al. (1964), Theirs (1965); and Theirs and Seed (1968)), few attempts (as seen in previous section) have been made to study the effect of frequency. In addition these studies are only in the range of 0-10 Hz which is significantly smaller than the 50-1000 Hz range which is involved in dynamic centrifuge tests.

It is evident that the effect of high frequency loading on soil behavior is dependent on the corresponding high strain rate loading (Section 2.0) and the degradation of cyclic strength during cyclic loading. Therefore, as a first approximation (Lefebvre, et al., 1987) the cyclic strength of a clay at a given frequency could be evaluated by first applying a correction for the strain rate effect to obtain the monotonic strength at the given frequency and then applying a degradation function to obtain the cyclic strength that can be mobilized for a given number of cycles.

Fo. example, in a 80 g centrifuge test, the strength of a given soil at 120 Hz and $\pm$ 5% cyclic strain amplitude could be obtained as follows:

$$\left.\begin{array}{l}\text{Corresponding monotonic}\\ \text{Strain Rate}\end{array}\right\} = \frac{2 \text{ x strain amplitude}}{1/2 \text{ x cyclic frequency}}$$

$$\dot{\epsilon} = \frac{2\,\epsilon_{cyc}}{1/2\ f}$$

$$= 4\,\epsilon_{cyc}\ f\%/\text{sec}$$

$$= 4 \times 5 \times 120 = 2400\%/\text{sec}$$

$$= 2400 \times 3600 = 8.64 \times 10^{6}\%/\text{hr}$$

From the extrapolation of Lefebvre, et al. (1987)'s results, the increase in shear strength ratio [(i.e. $c_u/(c_u$ at 1%/hr))] in the above case would be 1.65. For 20 cycles of loading, cyclic stress ratio (c_c/c_u at 1% hr) is 0.38 (Lee and Focht, 1975) where c_c - shear strength after cyclic loading. Combining the effect of strain rate and cyclic loading (Lefebvre, 1987) the resultant shear strength ratio (c_c/c_u at 1%

hr) would be (1.65 x 0.38) = 0.63. On the other hand, extrapolation of Cheng's (1980) results would give c_c/c_u at 1% hr as (2.15 x 0.38) = 0.82. Perloff, et al. (1963)'s results could not be used to extrapolate for the strain rates encountered in the centrifuge testing, as the values for parameters a, b, s, d and f reported in Table 4 are for the range of strain rates less than 100%/hr.

The above calculations are repeated for the prototype and the results are reported in Table 5 together with the model test results. It can be observed from the above table that the Cheng's (1980) results predict the model shear strength ratio to be 1.86 times greater than that of corresponding prototype while Lefebvre's (1987) results yield a value of 1.14. This large discrepancy is due to the fact that Cheng's (1980) effect of rate of strain on the shear strength relation show a drastic increase in strength ratio (1.0 to 2.2), for strain rates going from 10%/sec to 100%/sec.

Hence, the extrapolation of the existing results indicate that the undrained shear strength of a soil is increased by a factor of 1.14 to 1.86 (compared to prototype) for high frequency loading, depending on the corresponding monotonic strain rate. However, it should be noted that these factors are based on results obtained from results which may be susceptible to highly *non-uniform* samples as reported in section 2.2.

3.4 Centrifuge Case Studies

The centrifuge scaling laws can be verified directly by comparing prototype behavior to modelled behavior or indirectly by comparing modelled behavior to behavior predicted using experimentally measured soil properties with an analytical procedure. For dynamic soil behavior there is little detailed prototype experience which can be compared to model behavior. In order to overcome this difficulty, centrifuge modellers employ modelling of models, which is a modification of the direct verification (Schofield, 1981). A model tested at N g's is the prototype which is modelled by a 1/C scale model tested at CN g's where C is a factor. The scaling laws can be evaluated by comparing the measured behavior of the model-of-the-model to the measured behavior of the model.

Since the centrifuge scaling laws do not preserve identical strain rates in model and prototype, modelling of models may not give satisfactory comparison of strain and strength between models, due to high strain rates and frequency effects on soil behavior described in previous sections. But in the literature, there are case studies where modelling of models have shown to give convincing results to verify centrifuge scaling laws.

Schmidt (1978) obtained very satisfactory agreement between the model and prototype crater sizes resulting from explosions. Kutter, et al. (1988) also observed an excellent agreement in modelling of model experiments carried out on models of flexible shallow tunnels in dry sand subject to blast loading from nearby high energy explosives (fig. 23).

Lambe and Whitman (1982) modelled the upward propagation of shear waves through a 10.8 m thick horizontal sand stratum by models of two different sizes. The cyclic horizontal displacements and settlements measured on a 0.305 m high model tested at 35 g's are compared to the behavior measured on a 0.133 m high model tested at 80 g's. The tests were performed with dry sand and saturated sand. The agreement was better for settlements in dry sand tests and for cyclic shear strains and for settlements during saturated sand tests as shown in figs. 24-26.

For dry sand tests model-of-the-model cyclic shear strains were 50-70% higher than model values when prototype accelerations were greater than 0.18 g (fig. 27).

In the study of the behavior of clay embankments during and after earthquake loading, Kutter (1982) used the technique of modelling of models to test the scaling laws. Four models at scales (N = 35.6 and N = 80) differing by a factor of 2.25 were subjected to earthquakes with the basic earthquake frequency properly scaled. Small models (N = 80) are identified as A & B while large ones (N = 35.6) as C & D. However, during an earthquake, the amplitude of the cycles was not constant, and the way it varied depended on the model scale. In addition, the higher frequency components were not similar for the models at different scales. Therefore, the natural frequency and the earthquake induced permanent displacements of the large model (N = 35.6) and the small model (N = 80) were compared through some theory (Kutter, 1982).

In comparing the resonant frequencies, it was assumed that for models representing the same prototype that: 1) the resonant frequency of the models was only a function of the amount of permanent displacement caused by the earthquake, and 2) the resonant frequency is the frequency at which the first large peak occurs in the response amplification spectrum. The response amplification spectrum is defined as the spectrum of the ratio of the spectral response of the crest motion to the spectral response of the base motion.

When these assumptions were made it was found (Kutter, 1982) that the models that were tested at higher frequency and strain rate (N = 80) had about 30% lower natural frequency (in prototype terms) than the models tested at lower frequency and strain rate (N = 35.6). If rate effects were causing the difference, the N = 80 models should have a higher frequency than the N = 35.6 models.

The difference in resonant frequency was attributed to the difference in edge effects of the models tested at different scales. The N = 80 models had a larger aspect ratio (width to height) than the large models and hence, were less restrained by the boundaries. The large N = 35.6 models were given more support at the boundaries and consequently had a higher (prototype) resonant frequency.

Similarly the earthquake induced permanent displacements were compared (Kutter, 1982) using some theoretical predictions. If the theory takes account of the important differences in the base accelerations, then, even if the theory is not perfect, the observed model behavior should have the same relationship to predicted behavior that the model of the model does. Even if the theory is in error for analysis of the models, the same errors should apply to the analysis of the model of the model.

For each of the four sliding block predictions that were applied to predict permanent displacements (rigid-plastic, degrading rigid-plastic, viscoelastoplastic and degrading viscoelastoplastic) the large models and the small models of the models were seen to have the same relationship to predicted deformations (figs. 28 and 29). For each prediction type, the large models (C and D) fall within the same relatively narrow band that the small models A and B do. Even for prediction 1 (shown in fig. 28) where there is a poor correlation between predicted and measured displacements, there is a relatively narrow band in which the data for small and large models fall.

If soil strength increased at a modest rate of 10% per log cycle of strain rate, the factor of 2.25 in modelling scale would produce a difference in strength of 10% x log (2.25) = 3.5%. The effect of a 3.5% change in strength may not be discernible amongst the scatter in figs. 28 and 29. Some data in the literature (shown in fig. 9) have suggested that the rate effect may be as large as 50% per log cycle at strain rates above about 100%/s. If this did apply to the kaolin embankments, the difference in strength between the models at different scale would be 18%. The effect of an 18% change would appear in fig. 29 unless it was systematically counteracted by some other difference between the models.

Figures 30 and 31 showed that a 20% change in strength of an undamped elastoplastic sliding block would produce about a factor of four differences in predicted displacement for the first earthquakes on models B and D (BI and DI). Each of the solid curves in figs. 30 and 31 represent analyses with differences in strength of 20% (which happens to correspond with yield acceleration coefficients of 0.0926, 0.193 and 0.294). For a given prediction in fig. 29, the thickness of the band of data is about a factor of four and there is no apparent bias for small models to have a lower measured displacement than the large models.

If the strength of kaolin did increase at 50% per log cycle of strain rate, the difference in measured displacements of large and small models should be apparent in fig. 29. From this information Kutter (1982) concluded that the rate effects on strength seems to be significantly less than 50% per log cycle of strain rate.

The yield acceleration of the centrifuge models for the sliding block analyses was based on the back calculated strength determined from the static failure of model A during rapid spin up of the centrifuge. The strain rates developed during this static failure are difficult to quantify but it may be reasonably assumed that the rates were a couple orders of magnitude lower than those that occurred during the earthquakes. Since the strength values obtained in this manner produced predicted displacements (in fig. 29) within about a factor of two of the measured displacements, it may be inferred that the increase in the rate of strain (from that during static failure to that during earthquakes) does not affect the strength by more than about 10%. This argument, however, presupposes that the viscoelastoplastic sliding block analysis is correct. The viscoelastoplastic sliding block parameter study (figs. 30 and 31) showed that a 10% change in strength would produce approximately a factor of two difference in predicted displacement.

Moreover, the predictions based on the back calculated yield acceleration tend to slightly underestimate measured displacements (fig. 29). If the dynamic strength (at high strain rates) of soil was significantly larger than the strength during the static failure, predictions would be expected to overestimate measurements.

In conclusion, Kutter (1982) pointed out that the rate and frequency effects appear to have no significant effect on the results from centrifuge modeling. He ascribes the more important sources of error to the differences in edge effects between the large and small models and normal experimental errors (such as imperfect specimens, error in transducer calibrations, etc.).

4.0 CONCLUSION

A centrifuge model tested at n g's is subjected to strain rates n times larger than those in the corresponding prototype. It is known that the strength and stiffness of soils is affected by the rate of strain.

The literature review indicates that the behavior of dry sands is not significantly affected by the change of strain rate for either monotonic or cyclic loading.

For cohesive soils there is a significant influence of strain rate. For monotonic tests the shear strength appears to increase at about 10% for a factor of ten increase in strain rate (10% per log cycle) over a large range of strain rates.

Most of the data indicate that the shear strength increases faster than 10% per log cycle at the very high strain rates which may occur in dynamic centrifuge tests. Due to the difficulty in conducting the tests and maintaining sample uniformity in very high strain rate tests, this increase in rate effect is questionable.

During high frequency, high strain rate cyclic loading, similar results are observed. In stress controlled tests the samples tend to be stiffer and stronger as strain rate increases. In strain controlled tests, the samples are stiffer during the first few cycles, but the stiffness degrades more rapidly as frequency is increased. After a given number of cycles of a given strain amplitude, the static strength is lower for samples cycled at high frequency.

The rate effects in cohesive soils present a serious concern to centrifuge modelers. It may not be valid to use data from slow tests to interpret the model tests. Though this problem is more pronounced in centrifuge tests, prototype soils (during real earthquakes) are also subjected to strain rates which may exceed the rates applied in laboratory tests. The influence of loading rate should be accounted for in prototypes and in models.

If it is found that rate effects are small enough, they may be handled by accounting for them in the analysis of the test data or by counterbalancing the rate effects by using slightly weaker soil in the model than in the prototype. Some laboratory data suggests that rate effects become more significant at high rates of strain. Centrifuge studies involving blast loading and earthquake loading on clay embankments do not indicate the presence of excessive rate effects.

Rate effects are known to be significant for cohesive soils and the question of how to deal with them is still open. The magnitude of the rate effects is also uncertain at high strain rates due to questions regarding sample uniformity during high strain rate tests. With modern instrumentation and control technology better data should be forthcoming. For cohesionless soils, however, it appears that the effect of high strain rate, high frequency loading is small and the principles of dynamic centrifuge modeling are valid.

5. REFERENCES

1. Bjerrum, L., Simons, N. and Torblaa, I. (1958). "The Effect of Time on the Shear Strength of a Soft Marine Clay," Proceedings of the Brussels Conference on Earth Pressure Problems, Vol. I, pp. 148-158.

2. Brewer, J. H. (1972). "The Response of Cyclic Stress on a Normally Consolidated Saturated Clay," Ph.D Thesis, North Carolina State University, Raleigh.

3. Brown, S. F., Lashine, A. K. F. and Hyde, A. F. L. (1975). "Repeated Load Triaxial Testing of a Silty Clay," Geotechnique, Vol. 25, No. 1, pp. 95-114.

4. Carroll, W. F. (1988). "A fast Triaxial Shear Device," Geotchnical Testing Journal, ASTM, Vol. 11, No. 4, pp.276-280.

5. Casagrande, A. and Shannon, W. L., (1948a). "Research on Stress Deformation and Strength Characteristics of Soils and Soft Rocks under Transient Loading," Soil Mechanics Series No.31, Harvard University, Cambridge, MA.

6. Casagrande, A. and Shannon, W. L. (1948b). "Stress-Deformation and Strength Characteristics of Soils under Dynamic Loads," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Vol. V, pp. 29-34.

7. Casagrande, A. and Shannon, W. L. (1949). "Strength of Soils under Dynamic Loads," Transactions, ASCE, Vol. 114.

8. Cheng, R. Y. K. (1980). "Effect of Shearing Strain Rate on the Undrained Strength of Clays," Laboratory Shear Strength of Soils, ASTM, STP No. 740, pp. 243-253.

9. Collin, A. (1846). "Recherches Experimentales sur les Glissements Spontanes des Terrains Argileux," Carilian-Goeung, Paris.

10. Colp, J. L. (1965). "An Experimental Investigation of the Continuous Penetration of a Blunt Body into a Simulated Cohesionless Soils," Report SC-RR-65-260, Sandia Corporation, Albuquerque, New Mexico.

11. Converse, F. J. (1961). "Stress-Deformation Relations for Soft Saturated Silt under Low - Frequency Oscillating Direct Shear Forces," Soil Dynamics, ASTM, STP No. 305.

12. Crawford, C. B. (1959). " The Influence of Rate of Strain on Effective Stresses in a Sensitive Clay," ASTM, STP No. 254, pp. 36-61.

13. Drescher, A. (1966). "Effect of Abrupt Variation of the Strain Rate Upon Strength Characteristics of a Cohesive Soils," Bulletin de l'Academic Poloaise des Sciences, Sene de Sciences Techniques, Vol. 14, No. 5, pp. 5-12.

14. Glasstone, S., Laidler, K. and Eyring, H. (1941). "The Theory of Rate Processes," McGraw Hill, New York.

15. Graham, J., Crooks, J. H. A. and Bell, A. L. (1983). "Time Effects on the Stress-Strain Behavior of Soft Natural Clays." Geotechnique, Vol. 33, No. 3, pp. 327-340.

16. Healy, K. A. (1962). "Triaxial Tests upon Saturated Fine Silty Sand," Report No.11, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.

17. Healy, K. A. (1963). " The Dependence of Dialation in Sand on Rate of Shear Strain," Report No. 13, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.

18. Jarret, P. M. (1967). "Time Dependent Consoildation of a Sensitive Clay," Materials Research and Standards, ASTM, 7(7), pp. 300-304.

19. Khaffaf, J. H. (1978). "Weakening of Undrained Saturated Clays under Cyclic Triaxial Stress," Ph.D Thesis, University of Manchester, England.

20. Kutter, B. L. (1982), "Centrifugal Modeling of the Response of Clay Embankments to Eartquakes," Ph.D Thesis, Cambridge University, England.

21. Kutter, B. L., O'Leary, M., Thompson, P.Y. and Lather, R. (1988). "Gravity-Scaled Tests on Blast Induced Soil- Structure Interaction," Journal of Geotechnical Engineering, Vol. 114, No. 4, pp. 431-447.

22. Lambe, P. C. and Whitman, R. V. (1982). "Scaling for Earthquake Shaking Tests on a Centrifuge," Proceedings of the Soil Dynamics and Earthquake Engineering Conference, Southampton, pp. 367-378.

23. Lee, K. L. and Focht, J. A. Jr. (1975). "Cyclic Testing of Soil for Ocean Wave Loading Problems," Proceedings of the Offshore Technology Conference, Vol. 1, Paper No. 2183, pp. 343-354.

24. Lefebvre, G. and LeBoeuf, D. (1987). "Rate Effects and Cyclic Loading of Sensitive Clays," Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 5, pp. 476-489.

25. Mitchell, J. K. (1964). "Shearing Resistance of Soils as a Rate Process," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 90, No. SM1, pp. 29-61.

26. Mitchell, J. K. (1976). "Fundamentals of Soil Behavior," John Wiley & Sons Inc., New York.

27. Murayama, S. and Hata, B. (1957). "On the Effect of Remolding Clay," Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol. I, p. 80.

28. Parmelee, R. A., et. al. (1964). "Seismic Effects on Structures Supported on Piles Extending Through Deep Sensitive Clays," Report to California State Division of Highways, Insitute of Engineering Research, University of California, Berkeley, California.

29. Peacock, W. H. and Seed, H. B. (1968). "Sand Liquifaction under Cyclic Loading Simple Shear Conditions," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 94, No. SM3. pp. 689-708.

30. Perloff, W. H. Jr. and Osterberg, J. O. (1963). "The Effect of Strain Rate on the Undrained Shear Strength of Cohesive Soils," Proceedings of the Second Pan-American Conference on Soil Mechanics and Foundation Engineering, Ib/2, pp. 103-128.

31. Richardson, A. M. Jr. and Whitman, R. V. (1963). "Effect of Strain Rate upon Undrained Shear Resistance of a Saturated Remolded Fat Clay," Geotechnique, Vol. 13, pp. 310-324.

32. Schimming, B.B., Haas, H. J. and Saxe, H. C. (1966). " Study of Dynamic and Static Failure Envelopes," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 92, No. SM2, pp. 105-124.

33. Schmid, W. E. and Kitago, S. (1965). "Shear Strength of Clays and Safety Factors as a Function of Time," Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, pp. 345-349.

34. Schmidt, R. M. (1978). "Centrifuge Simulation of the JOHNIE BOY 500 ton Cratering Event," Proceedings of the Ninth Lunar Planet. Sci. Conference, pp. 3877-3889.

35. Schofield, A. N. (1981). "Dynamic and Earthquake Geotechnical Centrifuge Modeling," Proceedings of the International Conference on Recent Advances in Geotechnical Engineering and Soil Dynamics, St. Louis.

36. Seed, H. B. (1960). "Soil Strength During Earthquakes," Proceedings of the Second World Conference on Earthquake Engineering, Vol. I, p. 183.

37. Seed, H. B. and Chan, C. K. (1964). "Pulsating Load Tests on Samples of Clay and Silt from Anchorage, Alaska," Report on Anchorage Area Soil Studies, Alaska, to U.S. Army Engineer District, Anchorage, Alaska, Shannon and Wilson, Inc., Seattle, Washington.

38. Seed, H. B. and Chan, C. K. (1966). "Clay Strength under Earthquake Loading Conditions," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 92, No. SM2, pp. 53-78.

39. Seed, H. B. (1967). "Slope Stability During Earthquakes," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM4.

40. Sheriff, M. A.and Wu, M. J. (1971). "The Dynamic Behavior of Seattle Clays," Department of Civil Engineering, University of Washington, Seattle, Contract No. GC-2023, Soil Engineering Report 6.

41. Taylor, P. and Hughes, J. (1965). " Dynamic Properties of Foundation Sub-Soils as Determined from Laboratory Tests," Proceedings of the Third World Conference on Earthquake Engineering, New Zealand, pp. 196-211.

42. Theirs, G. R. (1965). "The Behavior of Saturated Clay under Seismic Loading Conditions," Ph.D Thesis, University of California at Berkeley, California.

43. Theirs, G. R. and Seed, H. B. (1968). "Cyclis Stress-Strain Characteristicsof Clay," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 94, No. SM2, pp. 555-569.

44. Theirs, G. R. and Seed, H. B. (1969). "Strength and Stress-Strain Characteristics of Clays Subjected to Seismic Loading Conditions," Vibration Effects of Earthquakes on Soils and Foundations, ASTM STP No. 450, pp. 3-56.

45. Vesic, A. S., Banks, D. C. and Woodward, J. M. (1965). "An Experimental Study of the Dynamic Bearing Capacity of Footings on Sand," Proceedings of the Sixth International Conference of the Soil Mechanics and Foundation Engineering, Vol. 2, pp. 209-213.

46. Vicat, (1833). "Recherches Experimentales sur les Phenomenes Physiques qui Precedent et Accompagnent la Rupture ou l'Affaissement d'une Certaine de Solides," Annales des Ponts et Chaussees, pp. 201-268.

47. Whitman, R. V. (1957). "The Behavior of Soils under Transient Loadings," Proceedings of the Fourth International Conference on Soil Mechanic and Foundation Engineering, Vol. I, pp. 207-210.

48. Whitman, R. V. (1960). "Some Considerations and Data Regarding the Shear Strength of Clays," ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, pp. 581-614.

49. Whitman, R. V. and Healy, K. A. (1962), "Shearing Resistance of Sands during Rapid Loadings," Report No. 9, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.

50. Whitman, R. V. and Healy, K. A. (1963). "Shear Strength of Sands during Rapid Loadings," Transactions, ASCE, Vol. 128, Part 1, pp. 1553-1594.

51. Whitman, R. V. and Lascher, U. (1965). "Footings for Protective Structures," Proceedings of the Symposium on Protective Structures for Civilian Populations, National Academy of Sciences - National Research Council, Washington D.C..

52. Whitman, R. V. (1970). "The Response of Soils to Dynamic Loadings; Report No. 26, Final Report," Contract Report No. 3-26, U.S. Army Engineer Waterways Experiment Station , Corps of Engineers, Vicksburg, Mississippi.

(Whitmann, 1970)

53. Yong, R. N. and Japp, R. D. (1967). "A Flow Law for Clays in Dynamic Compression," Proceedings of the Symposium on Wave Propagation and Dynamic Properties of Earth Materials, ASCE, Albuquerque, New Mexico.

54. Yong, R. N. and Japp, R. D. (1969). "Stress-Starin Behavior of Clays in Dynamic Compression," Vibration Effects of Earthquakes on Soils and Foundations, ASTM, STP No. 450, pp. 233-262.

Soil	PI %	PL %	w %	Chamber Pres-sure psi	Static Compressive Strength psi	Strain-Rate Effect*: At Low Stress (1/2 to 4%)	Strain-Rate Effect*: At Peak Stress
Medium-soft, slightly sensitive clay, undisturbed	24	26	27	0	10	2.0	4.0
Compacted silty sand	17	11	12	0	25	1.8	2.7
Normally consolidated, sensitive ocean sediment, undisturbed	63	49	92	0	0.3	2.0	2.4
Tough compacted fill	41	21	26	42	35	2.0	2.0
Slightly organic silty clay; undisturbed, saturated	21	22	35	0 85	22 54	1.6 1.7	1.9 1.7
Compacted silty sand	17	11	16	0	10	1.7	1.7
Plastic clay, remolded	27	38	44	0	15	1.6	1.8
Plastic clay, remolded	27	38	48	0	7	1.6	1.8
Plastic clay, remolded, saturated	38	24	30	60	36	1.6	1.6
Stiff dry clay, undisturbed	23	30	20	0 30	250 330	1.4 1.4	1.6 1.4
Slightly organic silty clay; undisturbed, saturated							1.6
Compacted silty clay (also sedimented specimens of same soil)			20 20	0 15	30 40		1.3 1.2
Compacted plastic clay	38	24	25	0 15	25 40		1.4 1.5
Compacted clay loam	23	22	21	0 30	13 15	1.5 1.5	2.5 1.7

* Ratio of resistance at 1000%/sec to resistance at 0.3%/sec.

Table 1. Summary of transient - loading triaxial tests on cohesive soils (Whitmann, 1970)

	$\Delta\epsilon$ < 1/2%	$\Delta\epsilon$ < 5%
Deviator stress	Increases	Increases
Pore pressure	Unaffected or slight increase	Decreases
σ'_1/σ'_3	Increases	Unaffected or slight decrease

Table 2. Effect of strain rate on the mechanical properties of soil at different strain levels (Richardson et. al. 1963).

w_L (%)	w_p (%)	I_p (%)	G	Clay, Fraction (%)
54.5	26.0	28.5	2.74	85

Table 3. Classification properties of Grundite (Perloff et. al. 1963)

a	b	c	d	f
1.67	0.116	0.258	-0.955	0.176

Table 4. Numerical values of constants in equations (Perloff et. al. 1963).

Table 5. Comparison of resultant shear strength ratios for centrifuge model and prototype samples using the results of Lefebvre (1987) and Cheng (1980).

	80 g model, 120 Hz and ± 5% strain amplitude Monotonic Strain Rate = 4 x 5 x 120 - 2400%/sec		Prototype, 1.5 Hz and ± 5% strain amplitude Monotonic Strain Rate = 4 x 5 x 1.5 = 30%/sec	
	c_u/c_u at 1%/hr	c_c/c_u at 1%/hr	c_u/c_u at 1%/hr	c_c/c_u at 1%/hr
Lefebvre (1987)	1.65	0.63	1.45	0.55
Cheng (1980)	2.15	0.82	1.15	0.44

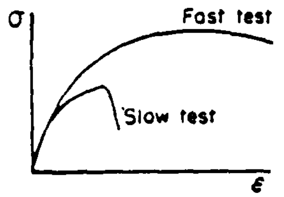
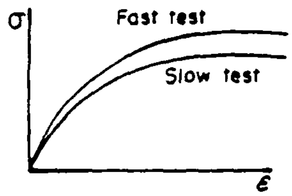
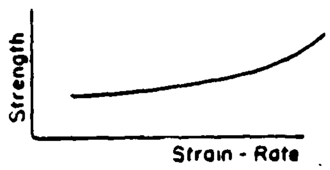
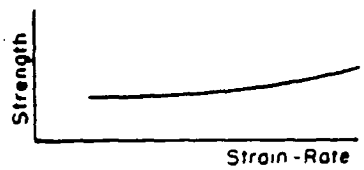
BRITTLE FAILURE	PLASTIC FAILURE
FAILURE BY SPLITTING OR PRONOUNCED FAILURE PLANES	FAILURE BY BULGING
Occurs where there are large negative pore pressures in unconfined compression tests: (1) Soils compacted dry of optimum water content. (2) Stiff saturated soils.	Occurs in triaxial tests with large chamber pressures, or where there are small negative pore pressures in unconfined compression tests: (1) Soils compacted wet of optimum water content. (2) Soft saturated soils.
STRAIN-AT-FAILURE AFFECTED BY STRAIN-RATE 	STRAIN AT FAILURE INDEPENDENT OF STRAIN-RATE 
LARGE STRAIN-RATE EFFECT 	MODERATE STRAIN-RATE EFFECT 

Figure 1. Hypothesis for difference in strain rate effect in different soils (Whitman, 1970)

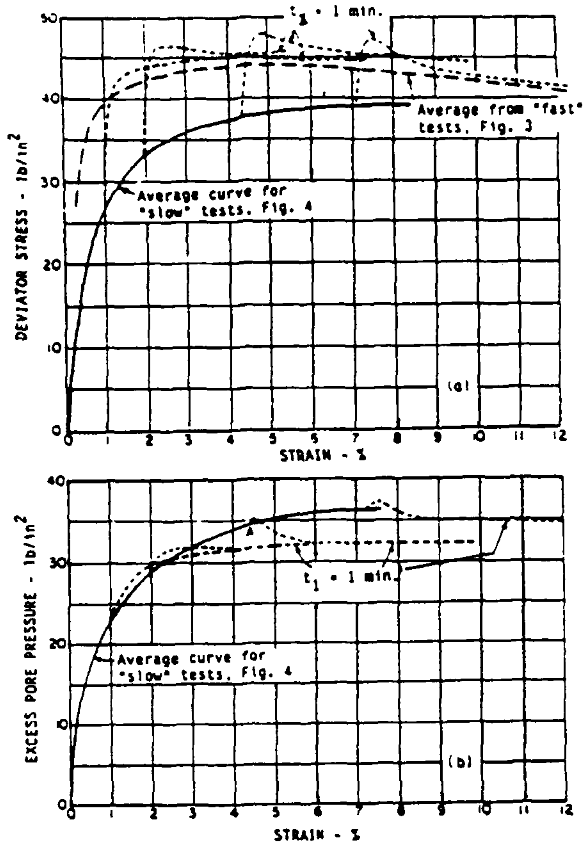


Figure 2. Deviator stress and pore pressure vs. starin from tests with change in strain rate for normally consolidated specimens (Richardson et. al. 1963).

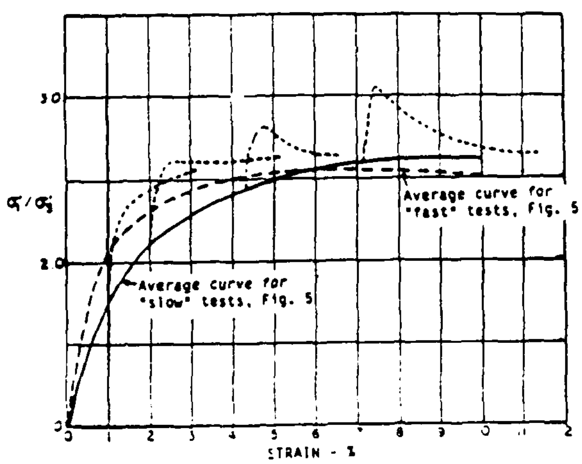


Figure 3. Obliquity ratio vs. strain from tests with change in strain rate for normally consolidated specimens (Richardson et. al. 1963).

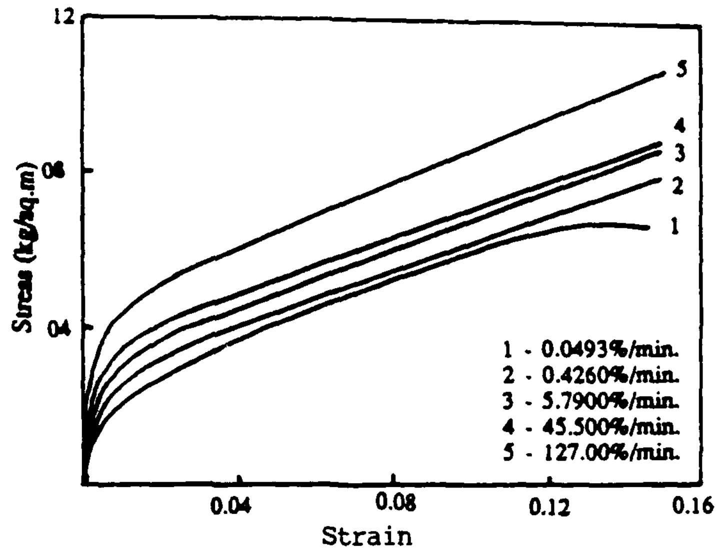


Figure 4. Deviator stress vs. strain from tests with different strain rates for remoulded soft plastic soil (Drescher, 1966).

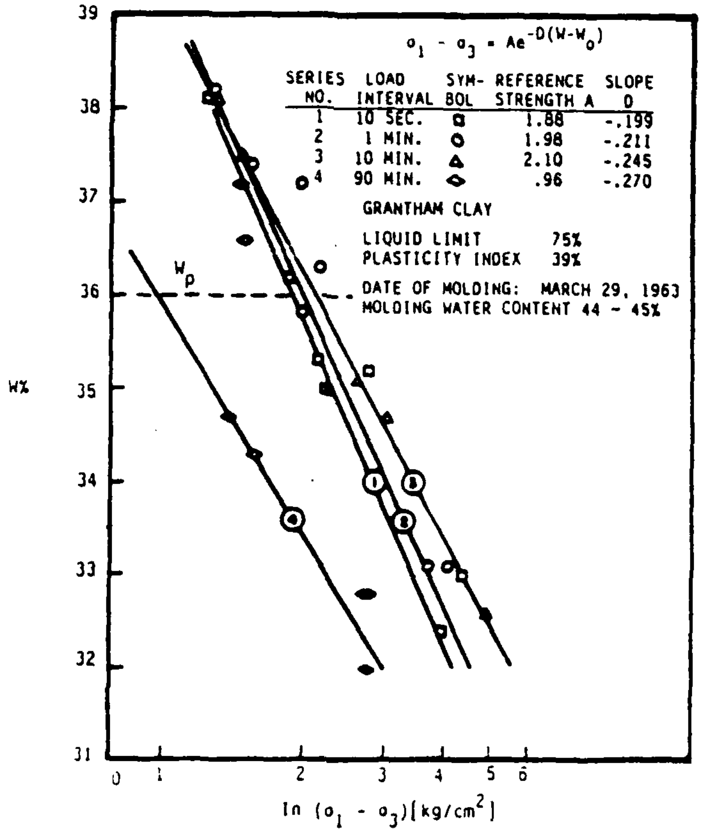


Figure 5. Water content vs. shear strength for different load intervals on Grantham clay (Schmid et. al. 1965).

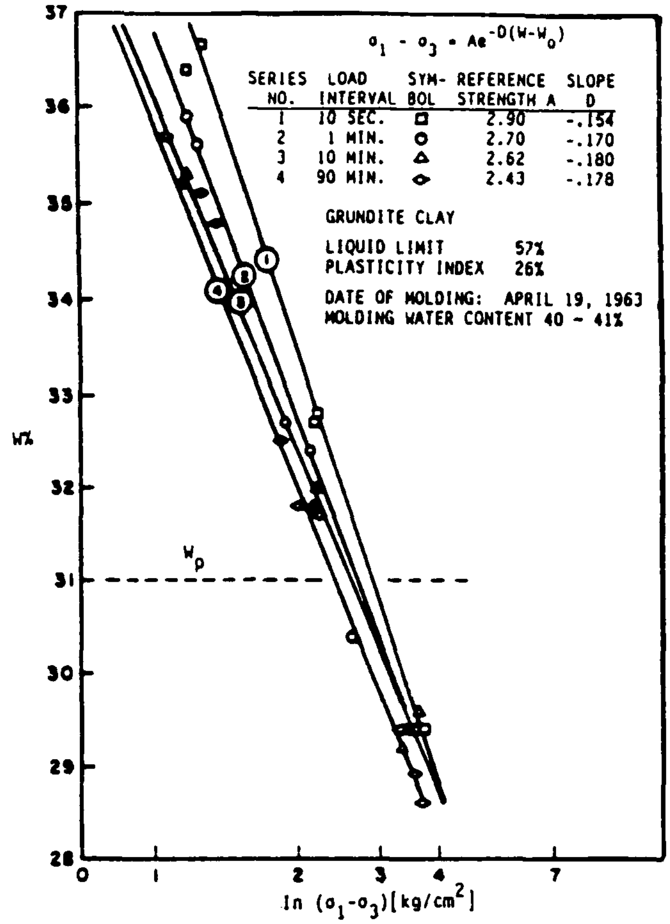


Figure 6. Water content vs. shear strength for different load intervals on Grundite clay (Schmid et. al. 1965).

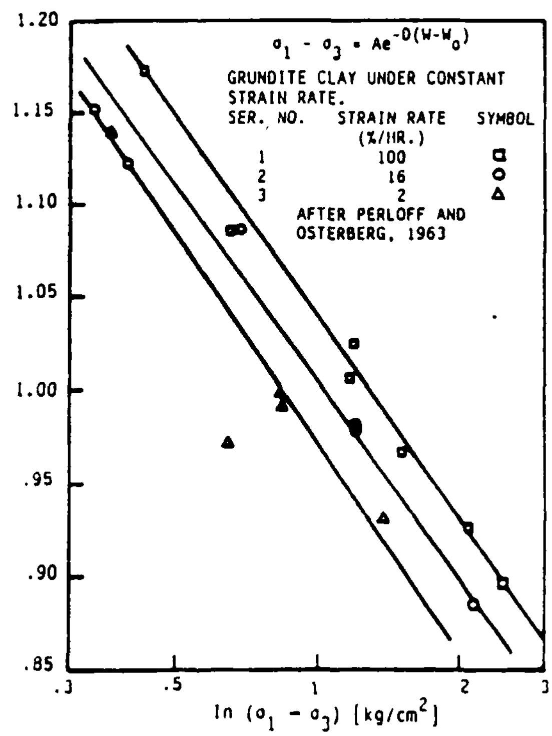


Figure 7. Water content vs. shear strength for different load intervals on Grundite clay after Perloff and Osterberg, 1962 (Schmid et. al. 1965).

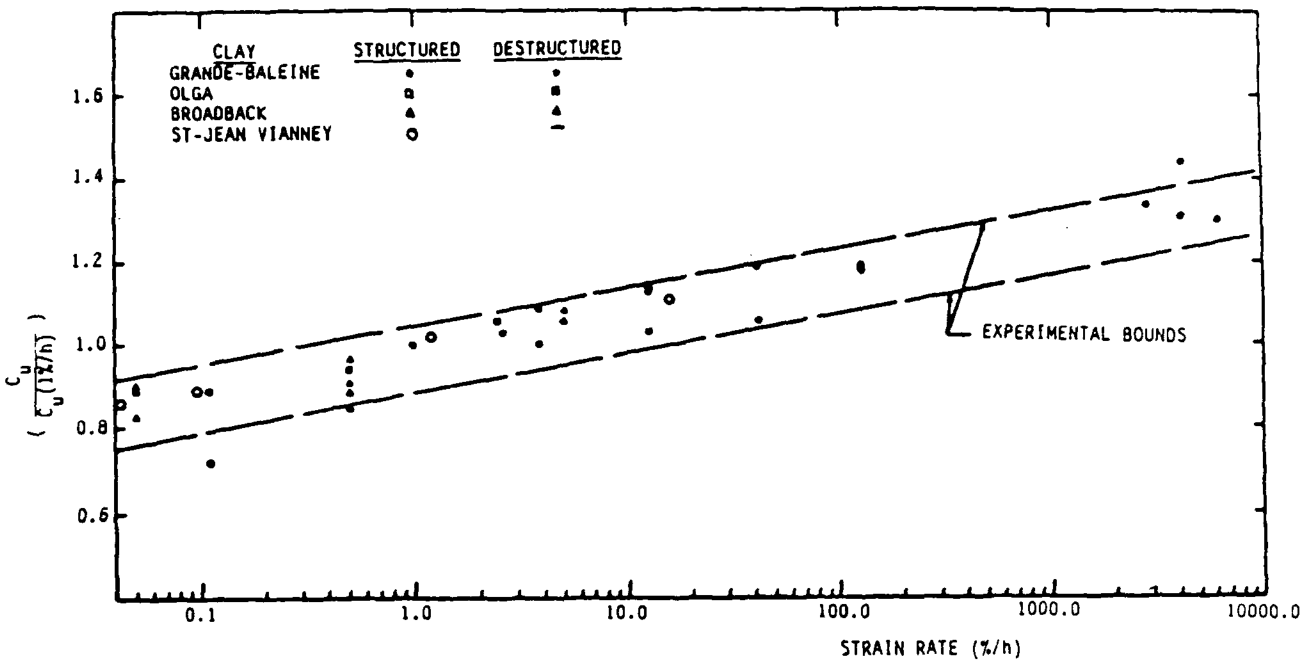


Figure 8. Change of undrained strength ratio, normalized to undrained strength ratio at $\varepsilon_1 = 1.0\%$/hour, with strain rate for all investigated clays (Lefebvre, 1987).

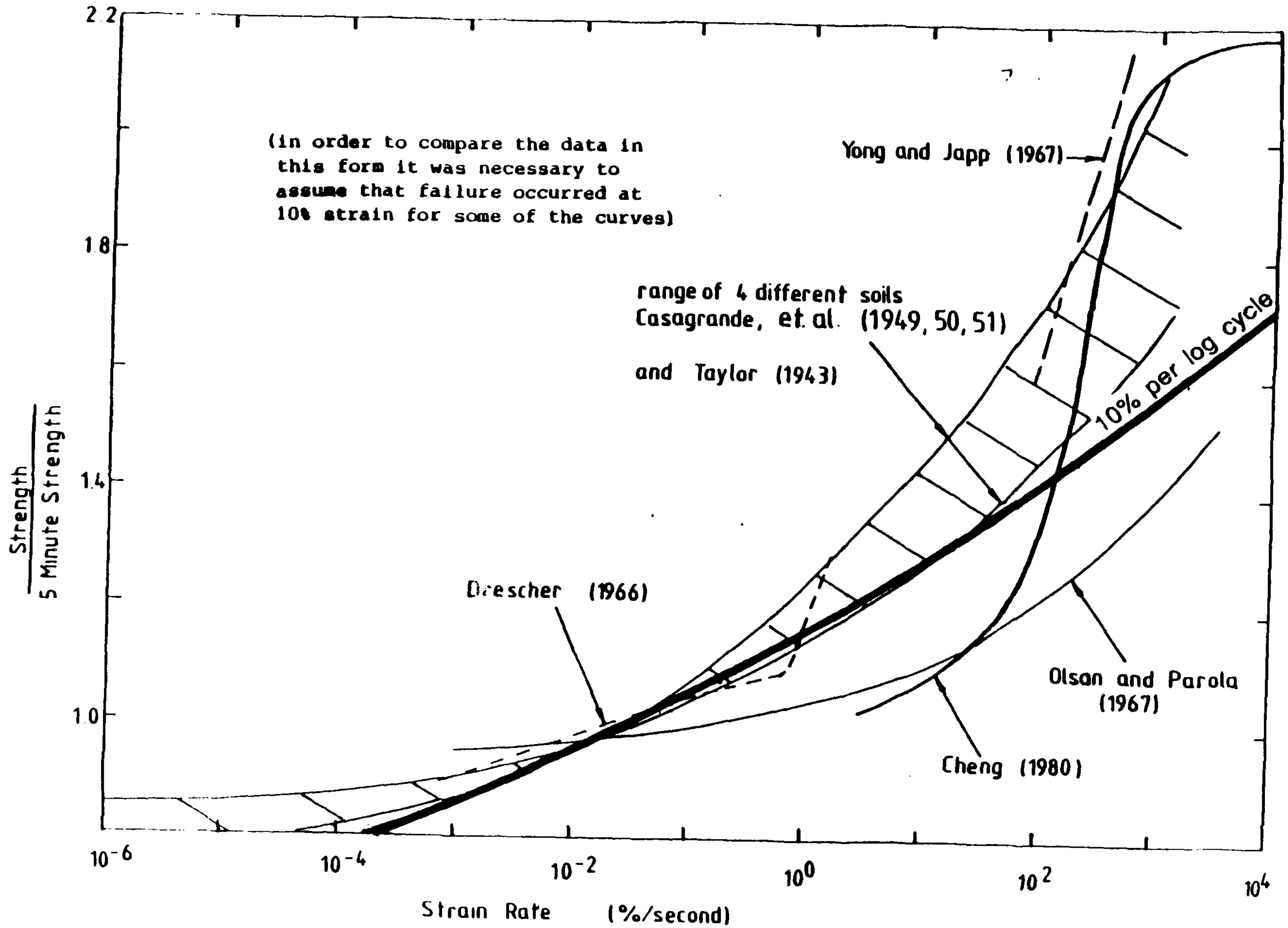


Figure 9. Comparision of some of the results reported by different authors regarding the effect of strain rate on the strength of long specimens (Kutter, 1982).

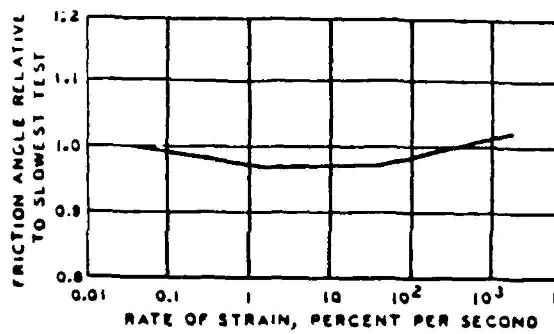


Figure 10. Composite average result for three dry sands from early tests at MIT (Whitman, 1970).

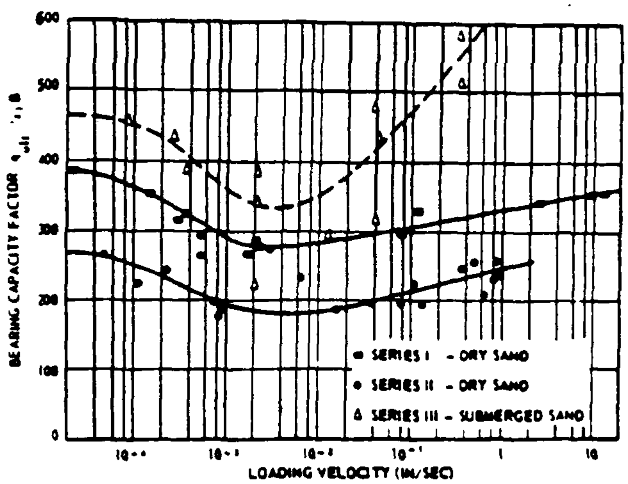


Figure 11. Bearing capacity factor vs. loading velocity (Vesic, 1965).

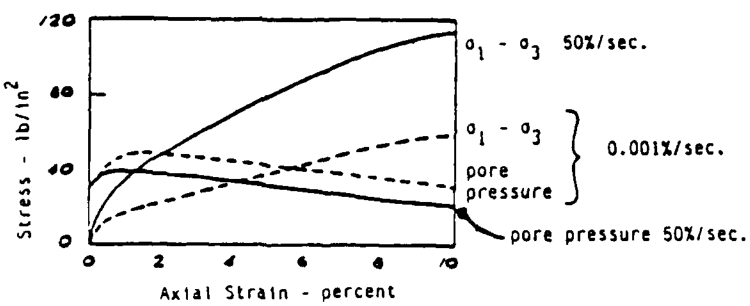


Figure 12. Stress strain curves for a loose, saturated fine sand (Whitmann, 1970)

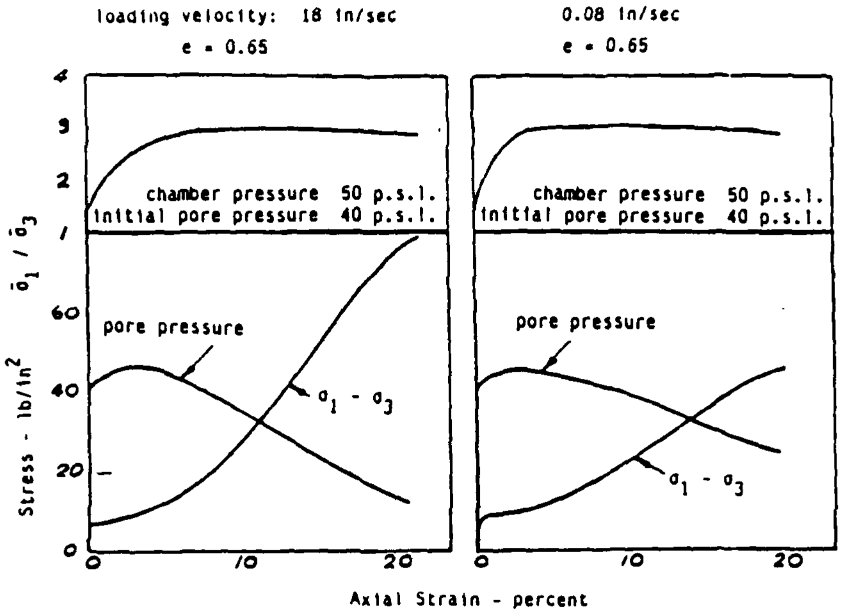


Figure 13. Stress strain curves for a loose, saturated Ottawa sand (Whitmann, 1970)

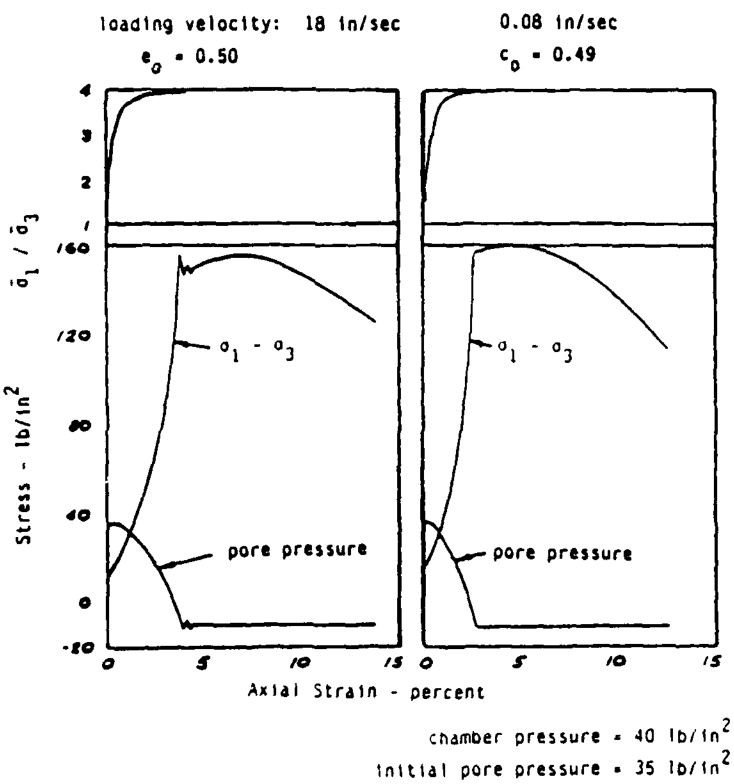


Figure 14. Stress strain curves for a dense, saturated Ottawa sand (Whitmann, 1970)

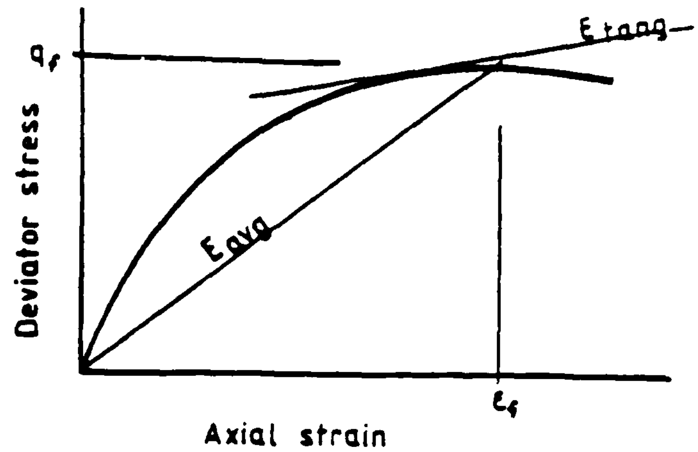


Figure 15. Definition of the avearge and tangent moduli for calculation of velocity of compression waves through triaxial specimens (Kutter, 1982).

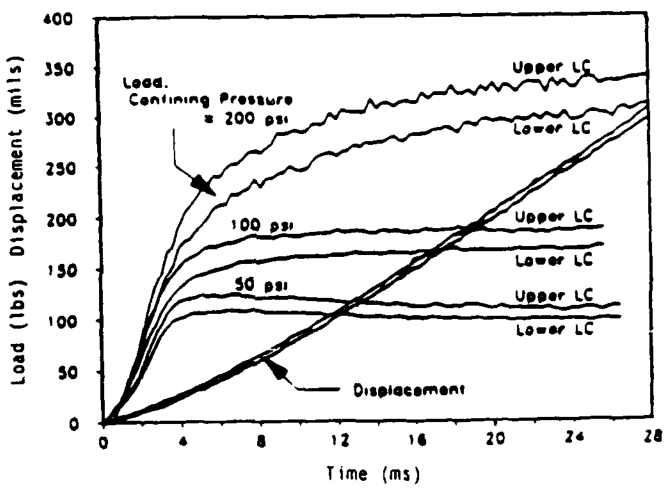


Figure 16. Load - Ddisplacement - time CARES - dry soil for test duration of 28 milli seconds(Carroll, 1988).

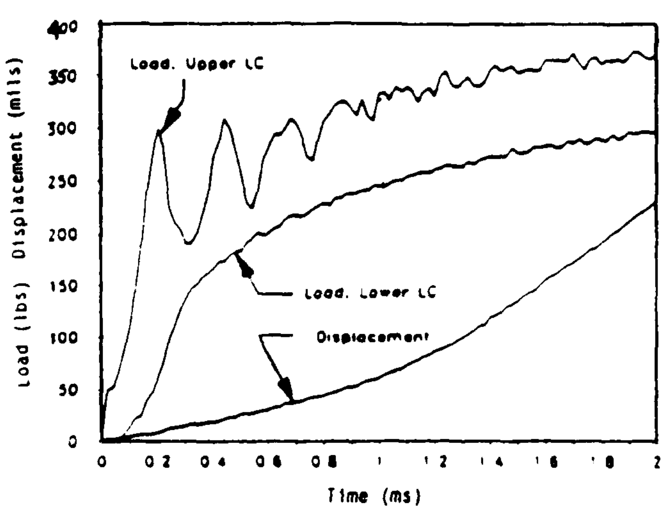


Figure 17. Load - Ddisplacement - time CARES - dry soil for test duration of 2 milli seconds and a confining pressure of 200 psi (Carroll, 1988).

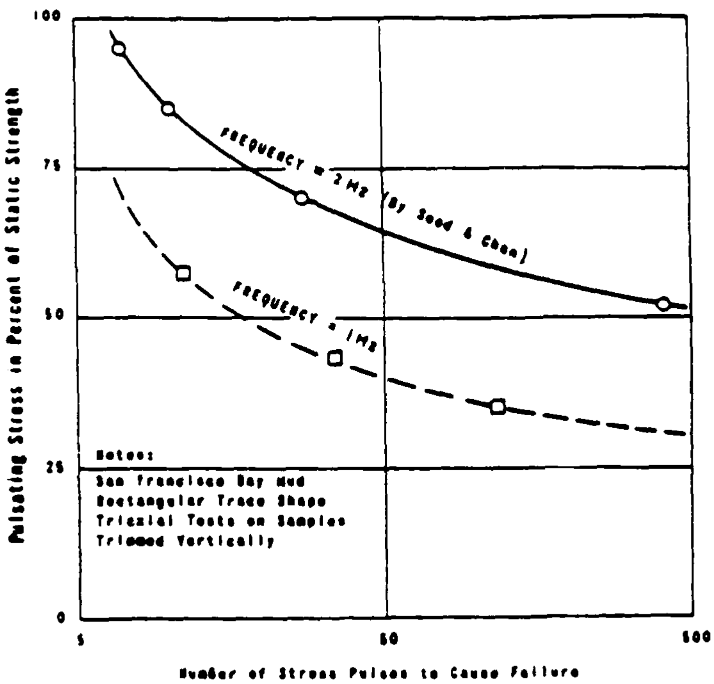


Figure 18. Effect of pulse frequency on number of pulses causing failure in pulsating load triaxial compression tests using normal trimming procedures for samples of San Francisco bay mud (Their and Seed, 1978).

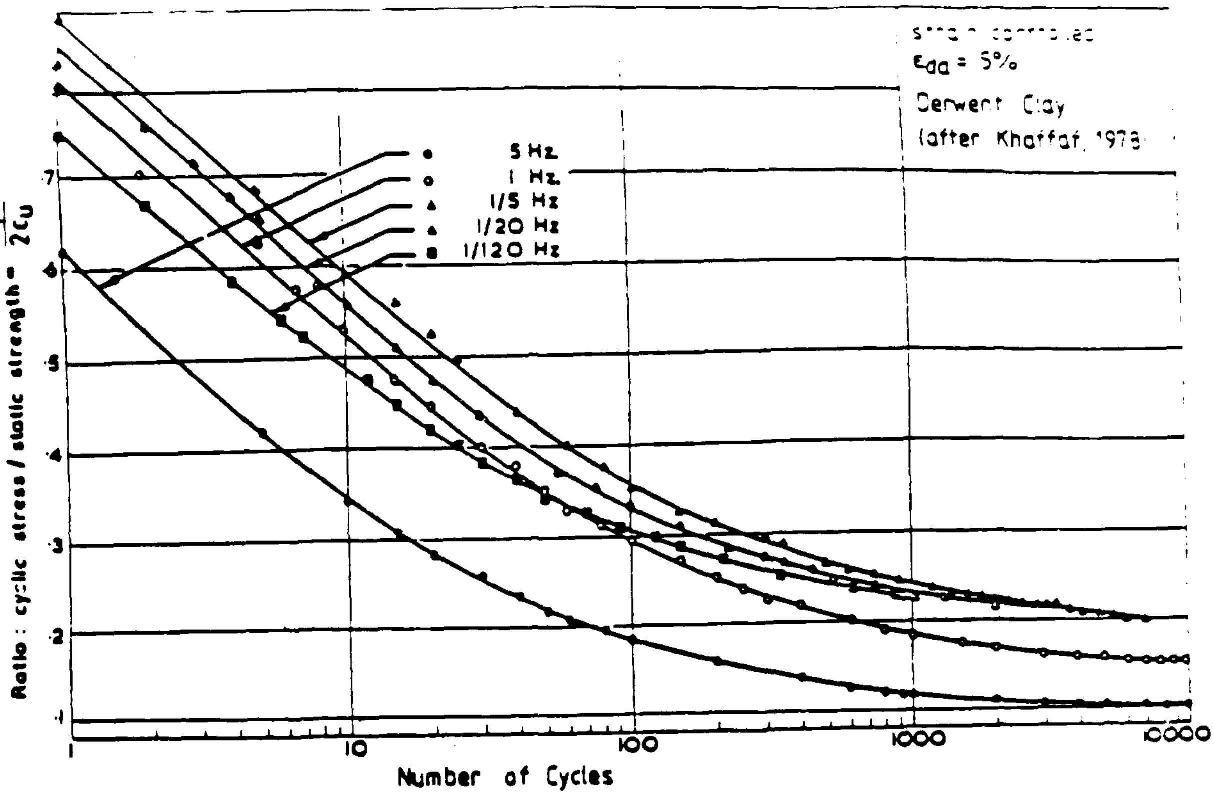


Figure 19. The effect of frequency on the relation between cyclic stress during a cycle and numer of cycles.

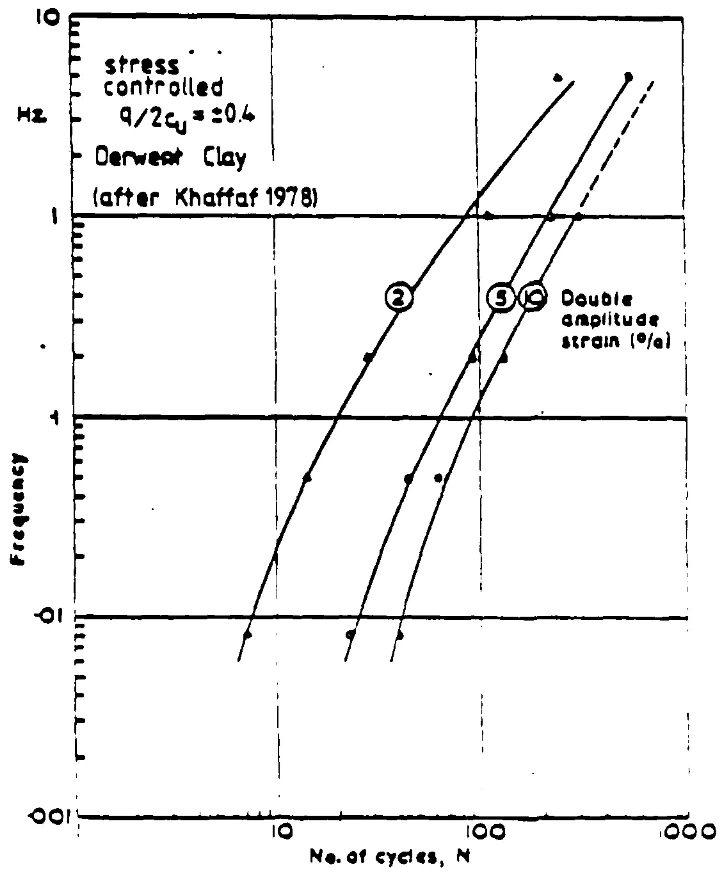


Figure 20. The effect of frequency on the number of cycles required to produce a given double amplitude strain.

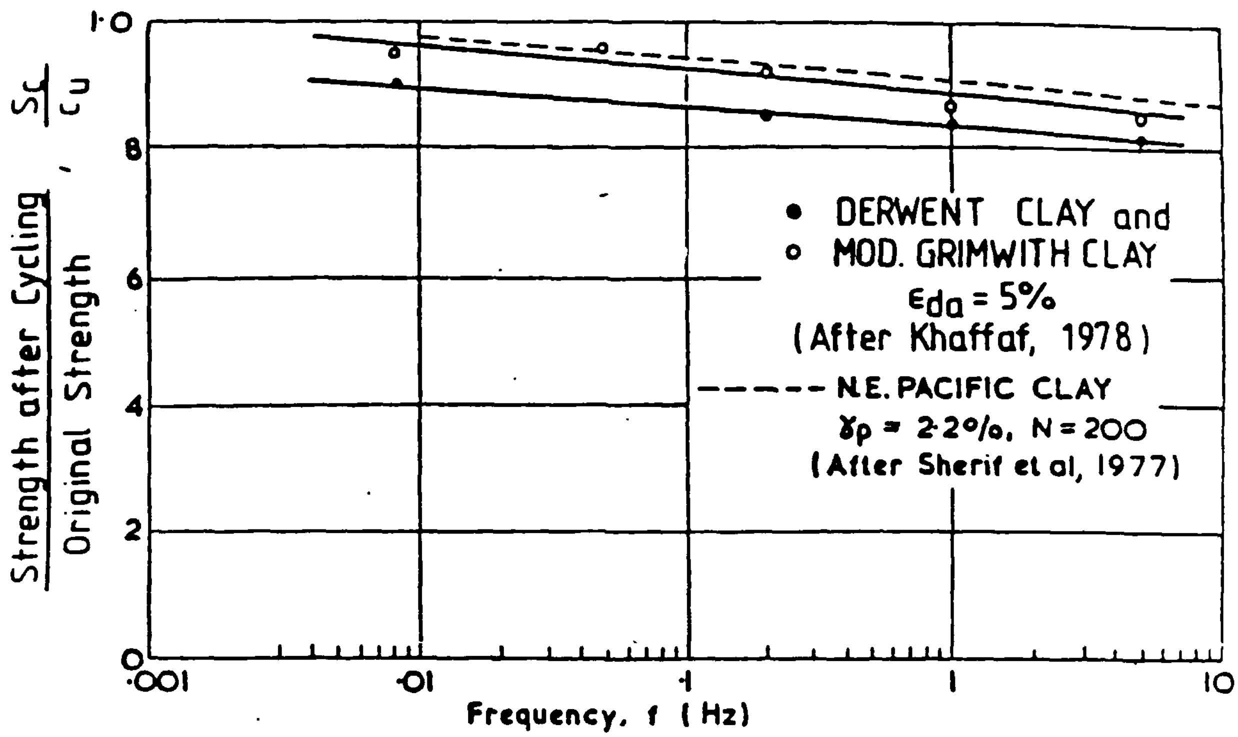


Figure 21. Relation between the degradation of strength due to strain controlled tests. Results from Sheriff et. al (1977) were obtained after 200 strain controlled cycles of amplitude 2.2%. The results from Khaffaf (1978) were obtained after the minimum stress level was achieved during strain controlled cyclic triaxial tests with peak strain of 5% (Khaffaf, 1978).

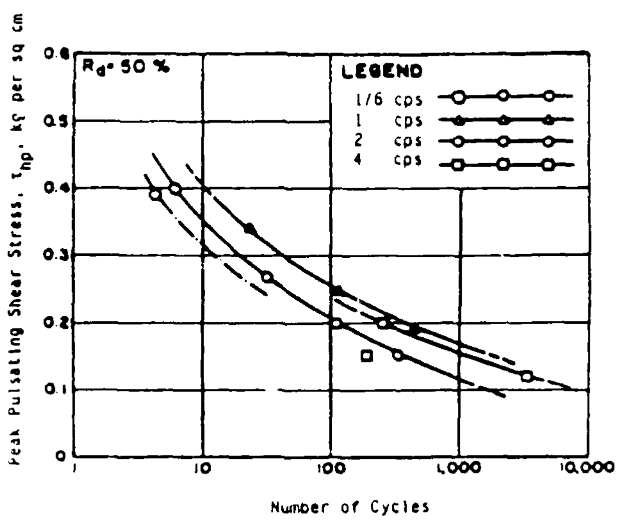


Figure22. Influence of frequency on undrained strength of loose monterey sand under cyclic loading - simple shear conditions (Peacock and Seed, 1968).

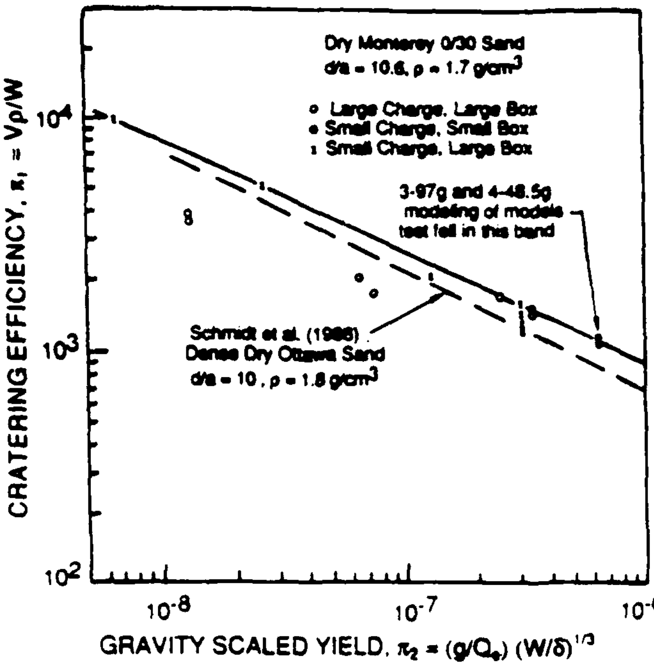


Figure 23. Cratering efficiency vs. gravity scaled yield (Kutter et. al. 1988).

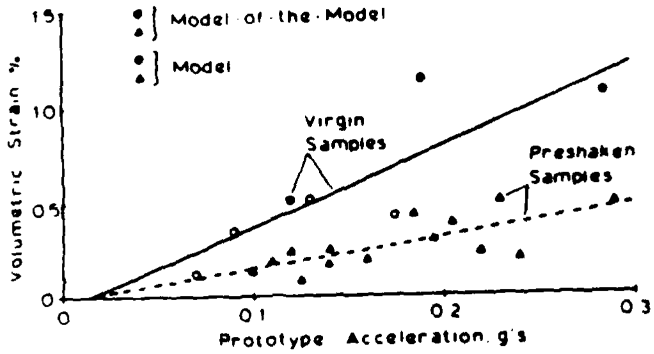


Figure 24. Volumetric strain vs. Prototype Base accelaration for dry sand tests (Lambe et. al. 1982).

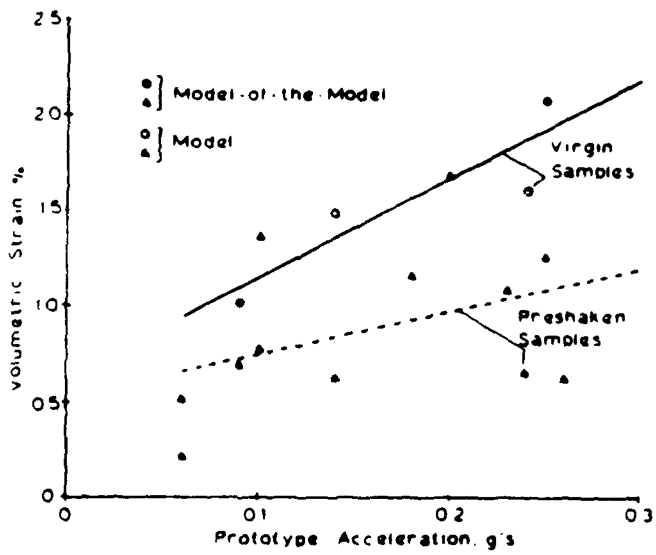


Figure 25. Volumetric Strain vs. Prototype Base accelaration for saturated sand tests (Lambe et. al. 1982).

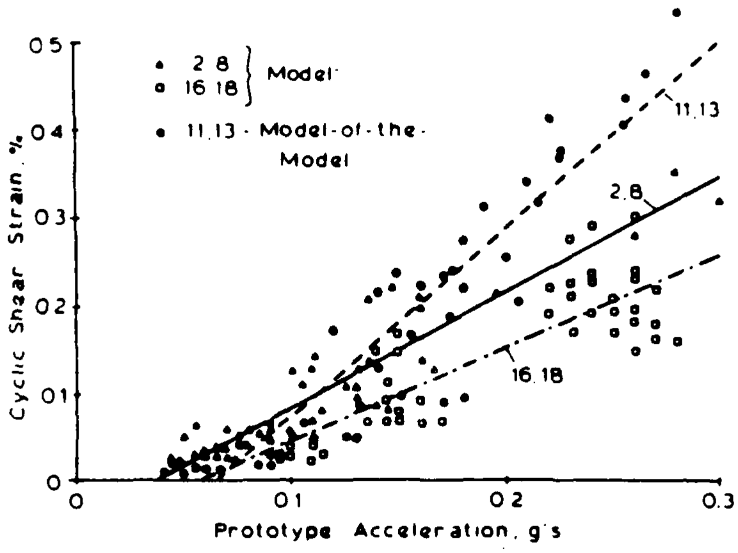


Figure 26. Cyclic Shear Strain vs Prototype Base accelaration for dry sand tests (Lambe et. al. 1982).

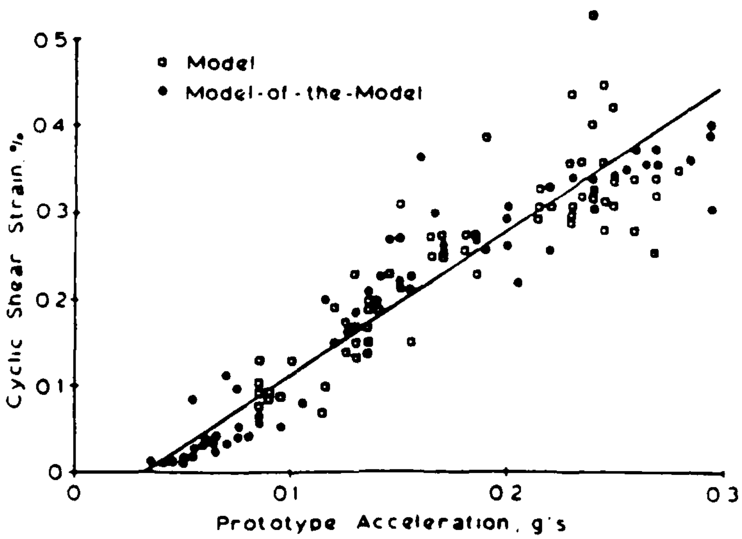


Figure 27. Cyclic Shear Strain vs Prototype Base accelaration for saturated sand tests (Lambe et. al. 1982).

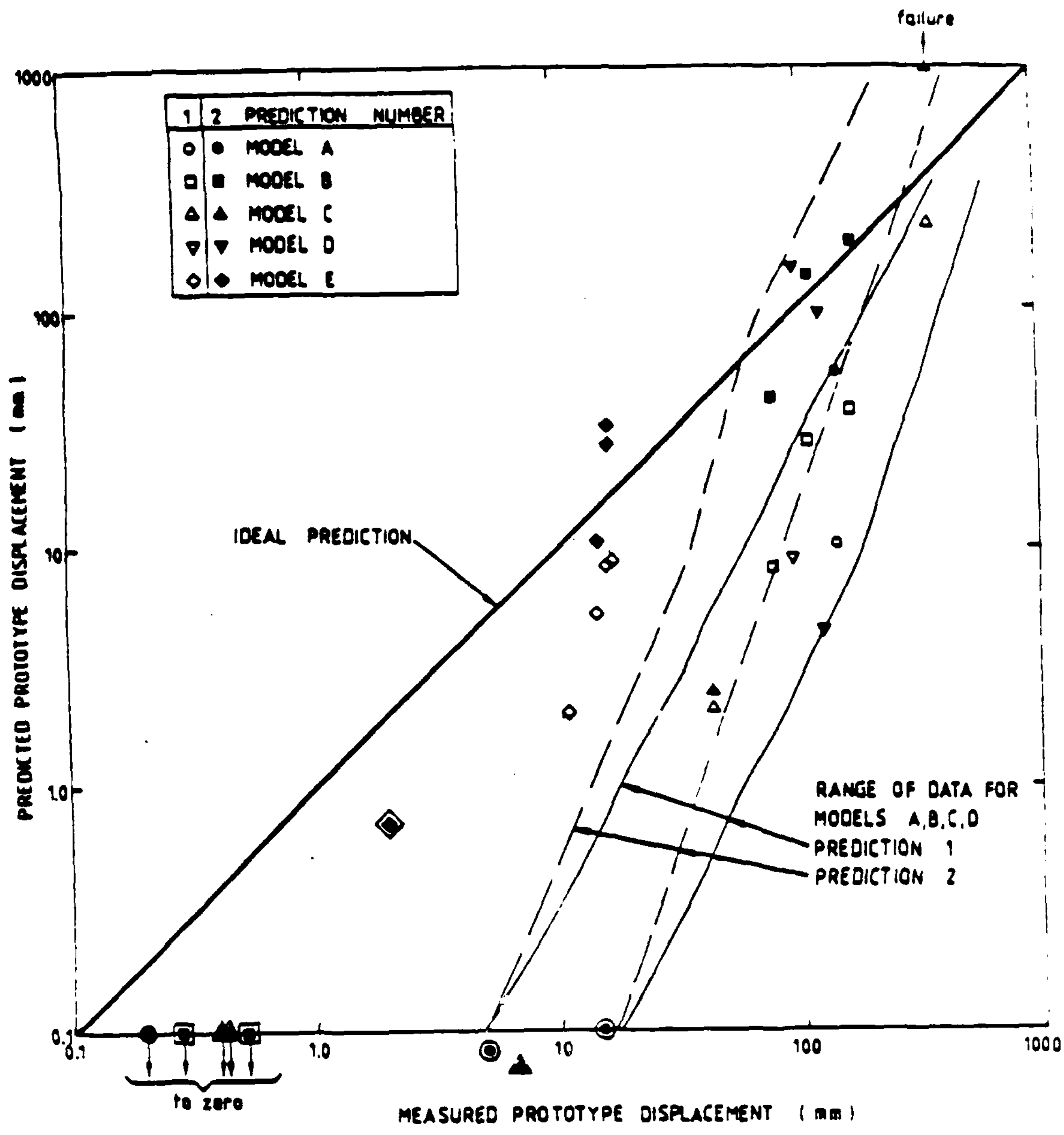


Figure 28. Comparision between predicted and measured displacements using rigid plastic sliding block analysis. Hollow points correspond to predictions that do not account for degradation of strength during shaking. If the theory agreed with the test results, the points would fall on the "ideal prediction" line (Kutter, 1982).

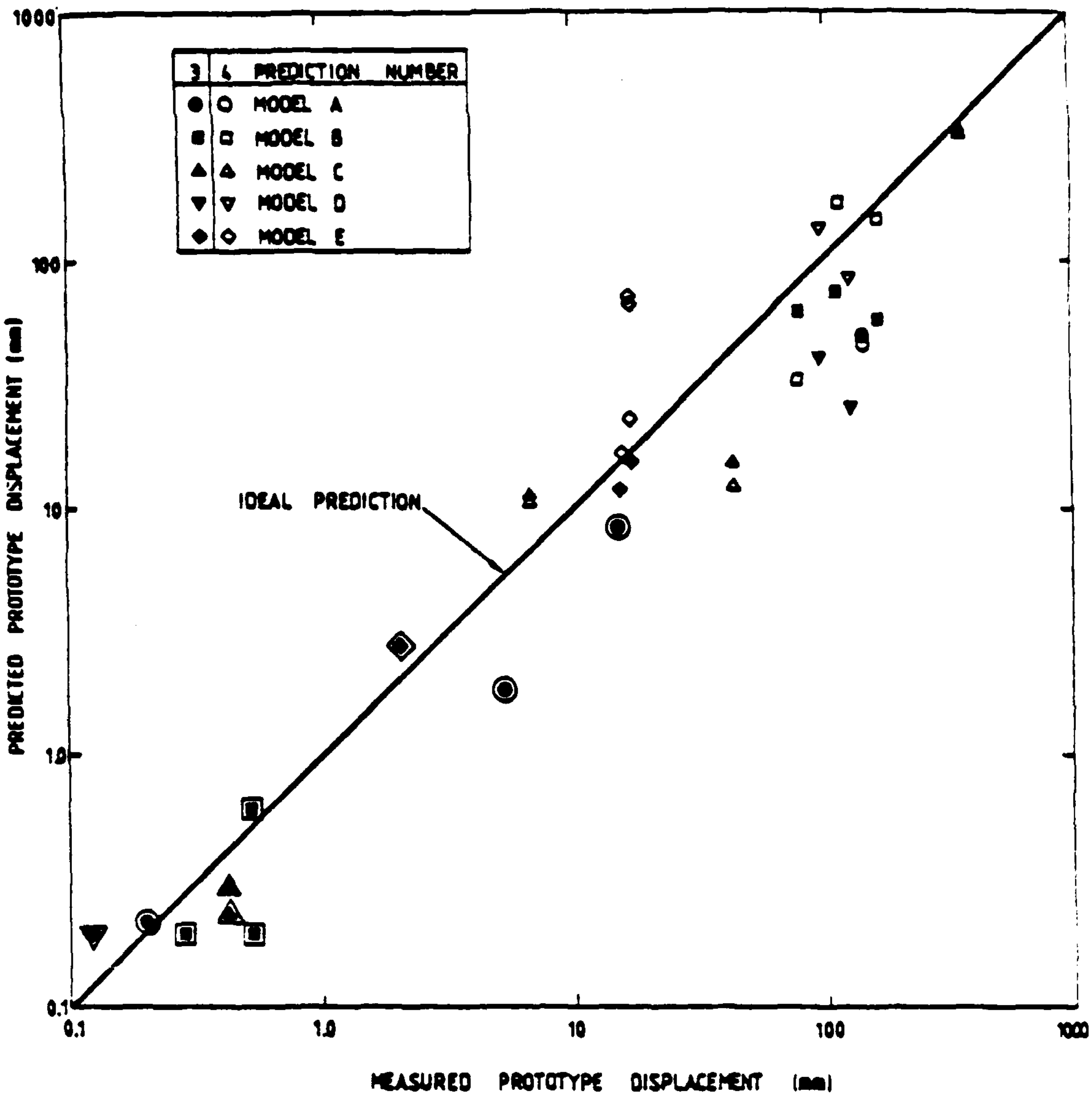


Figure 29. Relation between predicted and measured displacements using the new model. Closed symbols are for prediction 3 with no degradation. Open symbols are for prediction 4 which did incorporate degradation of strength and stiffness. If predictions exactly agreed with measurements from the centrifuge models, the points would fall on the "ideal Prediction" line (Kutter, 1982).

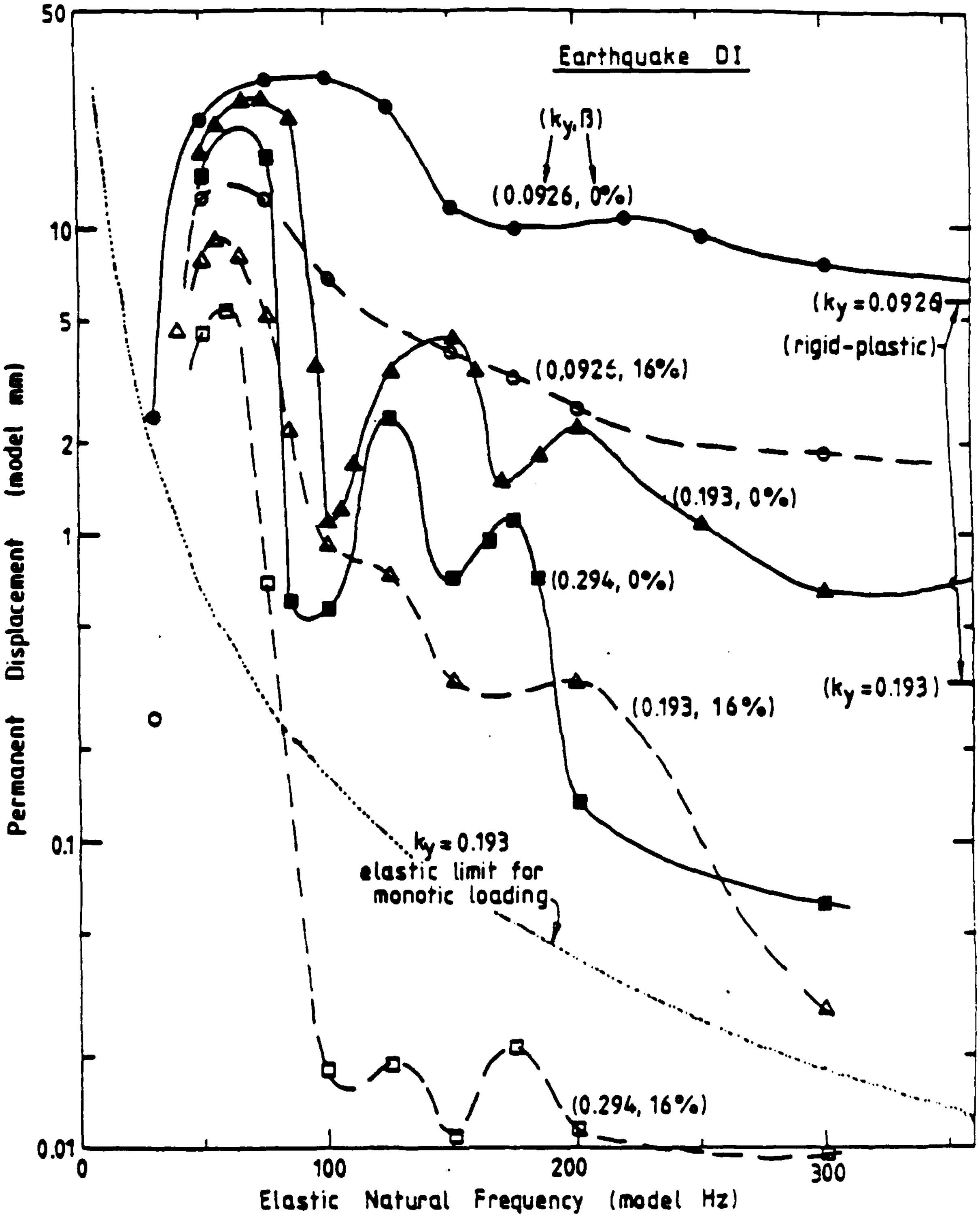


Figure 30. Effects of yield accelaration and damping on the relation between elastic natural frequency and the predicted displacement for model earthquake DI. (Linear elasto-plastic analysis, no degradation), (Kutter, 1982).

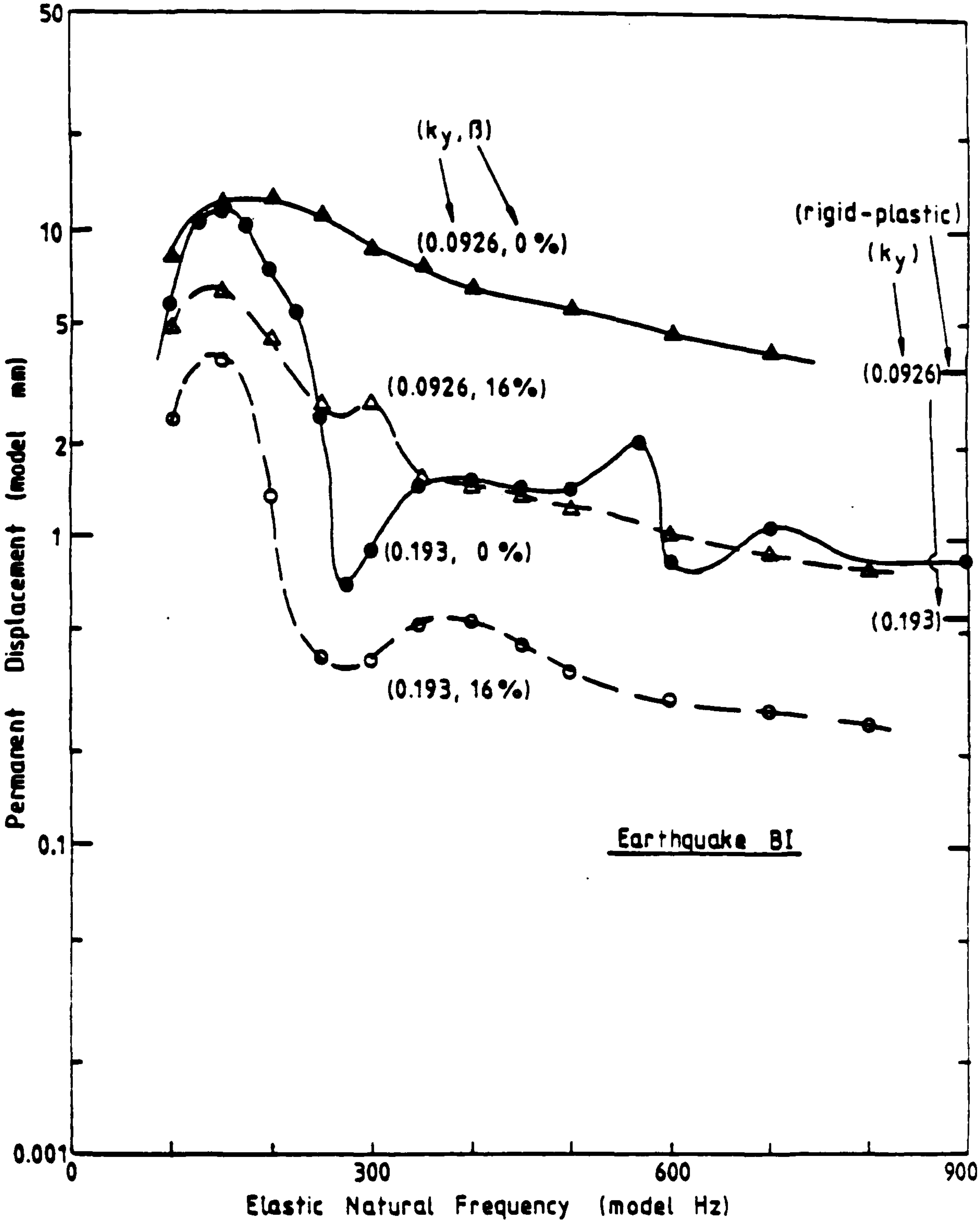


Figure 31. Effects of yield accelaration and damping on the relation between elastic natural frequency and the predicted displacement for model earthquake BI. (Linear elasto-plastic analysis, no degradation), (Kutter, 1982).

DISTRIBUTION LIST

AF AFWL NTE (Baladi). Kirtland AFB. NM
AFOSR NA. Bolling AFB. DC
ARMY CORPS OF ENGRS HQ. DAEN-ECE-D (Paavola). Washington. DC
ARMY EWES WES-SS (Kiger). Vicksburg. MS: WESIM-C (N. Radhadrishnan). Vicksburg. MS
CNR Arlington. VA
DEPT OF INTERIOR Minerals Mgmt Svc. Rsch Prgm Mgr. Reston. VA
DOT Dr Pin Tong. Cambridge. MA
DTIC Alexandria. VA
DTRCEN Code 1720. Bethesda. MD
GIDEP OIC. Corona. CA
NAVFACENGCOM Code 04BE (Wu). Alexandria. VA: Code 04R. Alexandria. VA
NAVFACENGCOM - CHES DIV. FPO-1PL. Washington. DC
NAVFACENGCOM - LANT DIV. Library. Norfolk. VA
NAVFACENGCOM - NORTH DIV. Code 04AL. Philadelphia. PA
NAVFACENGCOM - PAC DIV. Library. Pearl Harbor. HI
NAVFACENGCOM - SOUTH DIV. Library. Charleston. SC
NAVFACENGCOM - WEST DIV. Code 04A2.2 (Lib). San Bruno. CA
NORDA Code 360. Bay St. Louis. MS
NRL Code 4430 (Ramberg). Washington. DC
OCNR Code 00. Arlington. VA
PWC Code 101 (Library). Oakland. CA: Code 123-C. San Diego. CA: Code 420. Great Lakes. IL. Library (Code 134). Pearl Harbor. HI: Library. Guam. Mariana Islands: Library. Norfolk. VA: Library. Pensacola. FL: Library. Yokosuka. Japan: Tech Library. Subic Bay. RP
CATHOLIC UNIV of Am. CE Dept (Kim). Washington. DC
OREGON STATE UNIVERSITY CE Dept (Leonard). Corvallis. OR
RUTGERS UNIVERSITY CE Dept (Hanaor). Piscataway. NJ
STANFORD UNIVERSITY App Mech Div (Hughes). Stanford. CA
UNIVERSITY OF CALIFORNIA CE Dept (Herrmann). Davis. CA: CE Dept (Kutter). Davis. CA: CE Dept (Shen). Davis. CA: CE Dept (Taylor). Berkeley. CA: Geotech Model Cen (Cheney). Davis. CA: Mech Engrg Dept (Bayo). Santa Barbara. CA: Mech Engrg Dept (Tulin). Santa Barbara. CA
UNIVERSITY OF COLORADO CE Dept (Hon-Yim Ko). Boulder. CO
UNIVERSITY OF FLORIDA CE Dept (Townsend). Gainesville. FL
UNIVERSITY OF MARYLAND CE Dept (Goodings). College Park. MD: CE Dept (Wolde-Tinsae). College Park. MD
UNIVERSITY OF NEW MEXICO HL Schreyer. Albuquerque. NM
UNIVERSITY OF TEXAS CE Dept (Stokoe). Austin. TX
APPLIED RSCH ASSOC. INC Higgins. Albuquerque. NM
LOCKHEED Rsch Lab (B. Nour-Omid). Palo Alto. CA: Rsch Lab (S. Nour-Omid). Palo Alto. CA
SRI INTL Engrg Mech Dept (Grant). Menlo Park. CA: Engrg Mech Dept (Simons). Menlo Park. CA
TRW INC Crawford. Redondo Beach. CA: M Katona. San Bernardino. CA
UNITED KINGDOM Univ Coll Swansea (Zienkiewicz). Wales
WEIDLINGER ASSOC F.S. Wong. Palo Alto. CA
COX. J Davis. CA